JOURNAL

of the

AMERICAN CONCRETE INSTITUTE

Published monthly except July and August by the
American Concrete Institute under the authority of the Board of Direction

EDITED BY THE SECRETARY

with the direction of the PUBLICATIONS COMMITTEE: W. A. SLATER, chairman; P. H. BATES, ARTHUR R. LORD, J. C. PEARSON, A. N. TALBOT, BENJAMIN WILK, HARVEY WHIPPLE, Secretary.

Papers: Original papers, furthering the objects of the society, are invited. They should be addressed to the Secretary, and their consideration by the Publications Committee will be facilitated if: (1) a brief but comprehensive outline or summary precedes or at least accompanies the completed paper; (2) manuscript and photographs are available in duplicate or triplicate; (3) original tracings or other drawings, from which copies are readily made, are available.

Effort will be made to advise authors promptly of decisions as to availability for publication. Acceptance of a paper does not imply the acquiescence of critics (whose advice is sought prior to acceptance) or of the Institute in opinions which it expresses nor the acceptance of data or practice which it records. Institute authority attaches only to standards adopted as provided in the By-Laws. The publication of a committee report implies merely the cooperative effort of an appointed group within the Institute. Acceptance of a paper or report for publication does not commit the Program committee in reference to the annual convention.

Subscription price $12.50 per year payable in advance. To members, $10.00 per year, included in the annual dues.

Bound volumes 1 to 22 of *Proceedings of the American Concrete Institute* (1905 to 1926)—$8.00 each so far as available; Volumes 23, 24, 25 (1927-1929) $10.00 each. To members $5.00 each. Volume 26 (1930) consisting of JOURNAL issues 1-8, November 1929-June 1930, bound without *News Letter* pages to non-members and non-subscribers, $12.50. Subscribers and members may obtain the volume (in addition to their monthly issues) at the end of the volume year at $6.00, so long as a limited supply of over-copies permits. Members may insure availability of not more than two copies of each volume at a special price when they place their orders in advance of the first JOURNAL issue of each volume.

Publication address: 2970 West Grand Boulevard, Detroit, Michigan. Entered at the Post Office at Detroit, Michigan, as mail of the second class under the provisions of the Act of March 3, 1879.

Copyrighted, 1931
American Concrete Institute.

AMERICAN CONCRETE INSTITUTE

Founded 1905; incorporated 1906 (National Association of Cement Users); charter amended 1913. (American Concrete Institute.)

BOARD OF DIRECTION

President

D. A. ABRAMS (Term expires Feb. 1931)

Vice-Presidents

S. C. HOLLISTER (Term expires Feb. 1931)
ARTHUR R. LORD (Term expires Feb. 1932)

Secretary and Treasurer

HARVEY WHIPPLE (Term expires Feb. 1931)

Directors:

First District—R. B. YOUNG (Term expires Feb. 1932)
Second District—CLOYD M. CHAPMAN (Term expires Feb. 1932)
Third District—J. C. PEARSON (Term expires Feb. 1931)
Fourth District—P. H. BATES (Term expires Feb. 1931)
Fifth District—F. R. McMILLAN (Term expires Feb. 1931)
Sixth District—RAYMOND E. DAVIS (Term expires Feb. 1932)

Past Presidents

RICHARD L. HUMPHREY Deceased (1905-14)
LEONARD C. WASON (1915-16)
WILLIAM K. HATT (1917-19)
*HENRY C. TURNER (1920-21)
*WILLIAM P. ANDERSON (1922-23)
*A. E. LINDAU (1924-25)
*M. M. UPSON (1926-27)
*E. D. BOYER (1928-29)

*The five latest, living past presidents are members of the Board.

VOL. 2—No. 6 FEBRUARY 1931 (*Proceedings Vol. 27*)

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

News Letter

BEFORE YOU GO AND WHEN YOU GET THERE

Before You Go to Milwaukee

(1) Make your hotel reservation before Feb 20 (see rates elsewhere in these pages).

(2) Ask for a "Certificate" with your Railroad Ticket, so save half fare on your return trip.

(3) Familiarize yourself with papers and reports in the January and February JOURNALS which will be open to discussion at the convention after presentation in summary only.

When You Reach Milwaukee February 23 or 24

(1) Stow your baggage and then go to the 5th floor (Schroeder Hotel) and register—get a badge—deposit your railway certificate; meet the Milwaukee convention committee members—they will wear badges like this ——— Avail yourself of their courtesies; they want to help you make the most of your Milwaukee visit.

(2) If there are ladies in your party ask them to register—make themselves known to the hostesses of the Milwaukee committee.

(3) Buy dinner tickets—$2.00 (worth more) for Wednesday 6:30 p. m.

(4) Buy a luncheon ticket—$1.00—"get-together" right there on the fifth floor—at the Institute's opening opportunity for renewed acquaintance — at noon, Tuesday, February 24.

(5) Make the most of every opportunity the 27th annual convention affords—in the meetings, in the lobby.

TENTATIVE PROGRAM

(The Institute's 27th Annual Convention, beginning Tuesday, Feb. 24, is preceded by the Joint Meetings of the Concrete Masonry Association and the Wisconsin Concrete Products Association on Monday, with a dinner Monday evening.)

Tuesday, Feb. 24
Noon. Luncheon

$1.00 per Plate—Grand Ball Room—Fifth Floor Schroeder Hotel

2 p. m. First Session

Workability and Admixtures—General Discussion based on three papers:

(1) **A Study of the Flow Table and the Slump Test**
GEORGE A. SMITH AND SANFORD W. BENHAM, Concrete Engineer and Assistant Concrete Engineer respectively, Johns-Manville, Research Laboratories.

(2) **A Study of Slump and Flow of Concrete**
INGE LYSE AND W. R. JOHNSON, Assistant Engineer and former Assistant Engineer, respectively, Research Laboratory, Portland Cement Association
(Both the above published in the JOURNAL, January, 1931).

(3) **Admixtures and Workability of Concrete**
G. M. WILLIAMS, Professor of Civil Engineering, University of Saskatchewan.
(Published in the JOURNAL, February 1931).

Studies of Concrete Mixtures
S. C. HOLLISTER, Professor of Structural Engineering, Purdue University.

A Study of the Effect of Waterproofings on the Permeability of Concrete.
C. H. JUMPER, Bureau of Standards.

Making and Placing Concrete for the Calderwood Tunnels.
W. R. JOHNSON, Materials Testing Laboratory, Purdue University.

Tuesday, Feb. 24
8 p. m. Second Session

Design and Control of Concrete for Diablo Dam (February JOURNAL)
HERBERT F. FAULKNER, Assistant Engineer, City of Seattle and R. R. HUBBARD, Construction Superintendent, City of Seattle.

Some Long Time Tests of Concrete. (February JOURNAL).
M. O. WITHEY, Professor of Mechanics, University of Wisconsin.

Permeability of Gravel Concrete
PAUL T. NORTON, JR., Professor of Industrial Engineering, Virginia Polytechnic Institute and DAN H. PLETTA, Assistant Professor of Civil Engineering, University of South Dakota.

Properties of Mass Concrete. (January JOURNAL)
Report of Committee 108, RAYMOND E. DAVIS, Professor of Civil Engineering, University of California, Author Chairman and G. E. TROXEL, Assistant Professor of Civil Engineering, University of California, member of the committee, are joint authors of the report which has been reviewed by the committee as a whole.

The Durability of Concrete—A symposium by members of Committee 801.
F. R. MCMILLAN, Director of Research, Portland Cement Association,
R. B. YOUNG, Testing Engineer, Hydro-Electric Power Commission of Ontario.

Wednesday, Feb. 25
9:30 a. m. Third Session

Tests on Shrinkage of Concrete Masonry
P. M. WOODWORTH, Cement Products Bureau, Portland Cement Association.

A Comparison of Continuous with Batch Mixers in Plant Operation. (February JOURNAL).
BENJAMIN WILK, General Manager, Standard Building Products Co., Detroit.

Proposed "Recommended Practice for the Use of Color in Troweled Concrete Surfaces."
Report of Committee 408, W. D. M. ALLAN, Chairman.

Wednesday, Feb. 25
9:30 a. m. Third Session Continued

Proposed "Recommended Practice for the Manufacture of Concrete Masonry Units."
Report of Committee 708, P. M. WOODWORTH, Chairman.

Cast Stone Standards. (Proposed revision of tentative specifications).
Report of Committee 704, FRED WEIGEL, Chairman, C. G. WALKER, Secretary.

Proposed "Specifications for Concrete Burial Vaults."
Report of Committee 709. Presented by C. A. Wiepking, member of the Committee.

Appointment of Tellers to canvass Ballots for Officers and Directors.

2 p. m. Fourth Session

Field Practices in the Use of Multiple Sized Aggregates in Portland Cement Concrete Mixtures.
T. C. THEE, Highway Engineer, Bureau of Public Roads.
Discussion:
A. L. HAMBRECHT, State Engineer, Wisconsin State Highway Commission.
GEORGE W. LANGLEY, County Engineer, Sheboygan County, Wisconsin.

Determining the Characteristics of Concrete in the Mixer.
EMERY D. ROBERTS, Professor of Civil Engineering, Marquette University.

Design and Operation of Central Concrete plants.
Report of Committee 602, FRANK I. GINSBERG, Author Chairman.

Proposed "Specification for Ready-Mixed Concrete."
Report of Committee 503, MILES N. CLAIR, Author-Chairman.
This committee presented its first report in February, 1930 (*Proc.* Vol. 26, p. 467). Discussion was published in June (*Proc.* Vol. 26, p. 916). The present proposed specification was published November 1930 JOURNAL. Mr. Clair proposes to move the tentative adoption of the specification.

Wednesday, Feb. 25
6:30 p. m. 27th Annual Dinner

The dinner will be informal—a joint affair with the ENGINEERS' SOCIETY OF MILWAUKEE and the ASSOCIATED GENERAL CONTRACTORS OF MILWAUKEE. There will be good music. PRESIDENT ABRAMS will present the LEONARD C. WASON medal for "the most meritorious paper at the 26th Annual Convention" to I. E. BURKS, for his "Concreting Methods at Chute A Caron Dam." There will follow an "address" of very broad appeal—and an unusual entertainer, highly diverting.

Thursday, Feb. 26
9:30 a. m. Fifth Session

"Tentative Construction Specification for Concrete Work on Ordinary Buildings."

Report of Committee 502, ARTHUR R. LORD, Author-Chairman.

As published November 1329 (*Proc.* Vol. 26, p. 1) and amended at 25th annual convention, adopted as tentative (502 T 30). See further discussion, JOURNAL, November 1930. Mr. Lord proposes to move its submission to letter ballot for adoption as Standard 502.

Proposed "Specification for Supplying, Fabricating and Setting Reinforcing Steel on Ordinary Buildings."

Report of Committee 502, WILLIAM F. ZABRISKIE, Author-Chairman.

As published February 1930 (*Proc.* Vol. 26, p. 444) and discussed (*Proc.* Vol. 26, p. 910). Mr. Zabriskie proposes to ask the convention for its tentative adoption.

Proposed "Construction Specifications for Concrete Work on the Small Job."

Report of Committee 506, ARTHUR R. LORD, Author-Chairman.

As published JOURNAL, September 1930 and discussed January 1931. Mr. Lord proposes to move the tentative adoption of the specification.

Flow of Concrete Under the Action of Sustained Loads. (March JOURNAL).

RAYMOND E. DAVIS, Professor of Civil Engineering, and HARMER E. DAVIS, Instructor in Civil Engineering, University of California (presented as a part of the work assigned to Committee 109, Summary of Plastic Flow of which Professor Davis is Author-Chairman).

Thursday, Feb. 26 **9:30 a. m. Fifth Session Continued**	**Design of Concrete Buildings for Hurricane Exposures.** Report of Committee 308, ALBERT SMITH, Author-Chairman. **Reinforced Concrete Column Investigation.** (February and March JOURNALS). Progress Report of Committee 105, W. A. SLATER, Chairman.
2 p. m. Sixth Session	**Report of Tellers on election of Officers and Directors.** **President's Address** DUFF A. ABRAMS. **Amendment to By-Laws.** **Light Weight Aggregates and Light Weight Concrete**— A general discussion based on: (1) **Cinders as Concrete Aggregate.** (February JOURNAL). Report of Committee 203, EINAR CHRISTENSEN, Author-Chairman. (2) **Burned Clay and Shale Aggregates.** Report of Committee 204, GEORGE E. MCINTYRE, Author-Chairman. (3) **Use of Light Weight Concrete in Buildings.** (March JOURNAL). Report of Committee 406, FRANK A. RANDALL, Author-Chairman. **Unusual Features in the Design and Construction of a Twenty-Million Gallon Reservoir.** ARTHUR B. MORRILL, Assistant Engineer Filtration, Detroit Department of Water Supply. **The Use of Cement in Bulk.** Report of Committee 607, HERBERT COFFMAN, Author-Chairman.

SPECIAL ATTENTION TO EARLY HOTEL RESERVATIONS

MOSTLY, the hotels these days aren't crowded, but it is well to remember that as an Institute member you are entitled to special consideration if you make your room reservation early.

The management of the Schroeder Hotel (convention headquarters) has agreed to hold a block of rooms up to three days before the Institute meeting to meet the advance reservations of those attending the convention. After February 20, the Hotel Schroeder and other Milwaukee hotels will do the best they can to meet the demands of all comers. On or before February 20, your reservation at the Schroeder will be taken care of from the block reservations listed below. *Do not* send your reservations to the Secretary's office. Make your reservation direct to the hotel.

	One Person	Two Persons
125 Rooms, with bath.....	$3.50	$5.00
200 Rooms, with bath.....	4.00	6.00
50 Rooms, with bath.....	5.00	7.00
50 Rooms, with twin beds and bath for two.....		7.00

(Additional rooms with twin beds at $8.00, $9.00 and $10.00)

RAILROAD RATES AND SPECIAL ACCOMMODATIONS FOR MILWAUKEE MEETING

AGAIN INSTITUTE members may enjoy reduced railroad rates. When you buy your *going* ticket be sure to ask for a certificate, on account of the American Concrete Institute Convention. Deposit that certificate at the Institute's Registration desk, Schroeder Hotel, Milwaukee. If 150 or more certificates are validated you may, as in other years, ride home for half fare. Don't forget the certificate.

And if you are an Eastern member, don't forget that the Pennsylvania Railroad will run (in conjunction with the Chicago, Milwaukee, St. Paul & Pacific Railroad) through sleeping car service to Milwaukee, obviating a change of cars at Chicago. Departure will be from New York at 11 a. m. Monday, February 23, arriving Chicago February 24, 8 a. m., thence to Milwaukee over C. M. St. P. & P. (Train No. 5), arriving Milwaukee 10:20 a. m. Philadelphia passengers may board the special cars at 12:55 p. m. Monday and Pittsburgh passengers at 10:30 p. m.

FOR THE LADIES

MEMBERS of the Institute accompanied by their wives may safely leave the entertainment of the ladies to the Ladies Entertainment committee—L. S. Brodd, chairman.

First of all, the visiting ladies should register, with their husbands, so as to make their presence known. They will have opportunity to get acquainted with each other at a Tea in the Schroeder Hotel, Tuesday afternoon. In the evening there will be a theater party. Wednesday there will be a bridge-breakfast. In the evening they will find it worth while to join the men at the 27th Annual Dinner whose several special, attractive features have been the concern of the Institute's Milwaukee Convention Committee in cooperation with the Engineers' Society of Milwaukee and the Associated General Contractors of Milwaukee. We are led to believe that the dinner music will be of exceptional merit and following the dinner program special dance music will be provided.

Thursday the Ladies will have opportunities for sight-seeing—perhaps a candy factory, perhaps one where hosiery is made (both of national reputation).

I. E. Burks Wins Wason Medal

THE FIFTEENTH Leonard C. Wason Medal (founded in 1916 by the then president of the Institute as annual recognition of the year's "most meritorious paper") this year goes to I. E. Burks, Concrete Technician, Aluminum Company of America, Kenogami, Quebec, for his paper "Concreting Methods at the Chute a Caron Dam," published in February 1930.* It was selected by a committee of the Board of Direction as "the most meritorious paper" presented at the 1930 convention of the American Concrete Institute. Presentation will be made to Mr. Burks by President Abrams at the Institute's 27th Annual Dinner, Milwaukee, Feb. 25, at which the new medalist will be the Institute's guest of honor.

The Milwaukee Convention Committee

FOLLOWING is the general or executive body of the Milwaukee Convention Committee, which is working to make the 27th annual convention worth while in all those details in which only an active local group can arrange. (The entire committee organization was published in January.)

K. H. Talbot, Chairman
H. L. Foster, Vice Chairman
B. E. Brevik, Secretary
T. J. Baker
L. S. Brodd
D. R. Collins
A. W. Devos
Jack Franklin
E. E. Gillen
H. S. Greene
T. C. Hatton
W. P. Hirschberg
Morton Hunter
Robert Koehring
A. R. Morton
R. C. Newhouse
Prof. E. D. Roberts
J. P. Schwada
S. M. Siesel
F. M. Sloane
C. U. Smith
Maj. H. P. Tripp
Wm. Tubesing
F. W. Ullius
C. S. Whitney
C. A. Wiepking
Norman Wilson

*A. C. I. *Proc.* Vol. 26, p. 315.

Membership Honor Roll

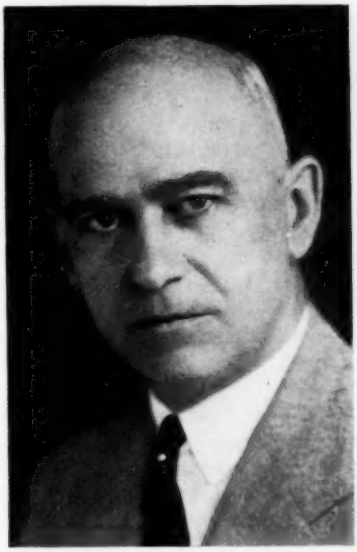
THE 1931 MEMBERSHIP Honor Roll has closed—total enrollment 16, each of them having sponsored three or more new members in the year ended February 1, 1931. The three leaders in the following list will receive special awards—W. D. M. Allan, who sponsored 21 new members; Prof. Raymond E. Davis, 10; F. R. McMillan, 9.

Besides the sixteen members who sponsored the requisite three, sixteen members sponsored two each—while eighty-seven are credited with one each.

W. D. M. Allan	21
Raymond E. Davis	10
F. R. McMillan	9
Miles N. Clair	5
P. G. Gauthier	5
D. A. Abrams	4
H. J. Gilkey	4
F. I. Ginsberg	4
M. M. Upson	4
Einar Christensen	3
Herbert Coffman	3
J. C. Pearson	3
F. E. Richart	3
Hale Sutherland	3
H. C. Turner	3
W. F. Way	3

*Fellows of the Institute**

1—DUFF A. ABRAMS



MANY YEARS AGO—having studied concrete one year, taught it the next—I decided to learn something about the subject by the experimental route. Talbot graciously permitted me to attach my sled to his cutter and we were off on a dizzy race—be careful where you hitch on for a ride! In those early days there was an Associate—not an Associate *Professor*, but just a plain Associate—in the T. and A. M. Dept., at Illinois, whose job was principally to manufacture all the specimens that the rest of us tested. He bore the strange name of Duff—not an earned nickname but merely the name wished on him by his parents. Everybody in the Laboratory liked him. Even I called him "Abe" from our first meeting, and it gave me no such thrill as it does today. When not making my specimens, or a host of others, this chap amused himself by sticking short bars in small concrete cylinders and then pulling them out on a small machine over in one corner of the laboratory. How should *I* know that when he was through, no one would think seriously of making more bond tests for twenty years? That was Duff's way of doing things—not exactly secretively, but quietly—the way the Chinese celebrate the Fourth of July.

After his famous Bond Bulletin appeared—impressive for size even if you have never opened the covers, for Duff insisted on more than a "shirt tail full of data" even then—people in faraway Chicago began to take notice of Duff and the bright lights of stardom began to call. Not that we considered it much of a promotion to make tests for a commercial bunch of cement manufacturers. But he became a full Professor at Lewis Institute and that adequately explained his eagerness to go to such a dead-head job. He began to accumulate another batch of data—ten thousand tests, twenty thousand, thirty, forty, fifty thousand—and never a bulletin issued from Chicago. He was just quietly making concrete cylinders, crushing them to bits, using the fragments to fill in Chicago's lake shore. That didn't seem like especially productive Research and we began to wonder just when Duff would be back looking for his old job at Urbana.

*A special order of distinction to which nominations are made from among his friends and associates in the Institute work by the author of a series of biographical sketches, of which this is the first. EDITOR.

And then, the thunderbolt! Move over, Professor Talbot, Dr. Hatt, Dean Turneaure, and make room on the intellectual throne for a newcomer who has made all the old art of concrete proportioning obsolete over night! *Not mixes nor finely disputed grading, but water-cement ratio determines the strength of workable concrete!* Old theories and facts had to be re-interpreted in the light of a newly discovered *law*. Now the bulletins poured forth from Chicago, tying up every property of concrete so hard and fast to the new law that I doubt we shall ever pry them loose. Things we could never understand became crystal clear. Concrete making became a science.

And in the process a great Laboratory was built up, that remains as a monument to Abrams. To have directed a commercial undertaking into the paths of pure research was not the least of "Abie's" accomplishments. So long as water-cement ratio determines concrete strength, the Materials Testing Laboratory of the Portland Cement Association must work for scientific results primarily—for commercial advantage only secondarily. It's creator has gone; in another of those sudden, hard-to-understand moves. He is again silently amassing another mountain of data. To what end? I do not know now, any more than I did in 1914. I know that he alone has made concrete that crushes at more than 50,000 lb. per sq. in. in twenty-four hours. But he is still mulling over the new data, continuing his endless round of tests, far from satisfied. His poise cannot be disturbed by external pressure—his whole history attests to that. In due time, as he reckons it, we shall know what the President of the Institute has in store for us.

—ARTHUR R. LORD

OF INTEREST IN MILWAUKEE

THE MILWAUKEE CONVENTION Committee, through C. U. Smith, in charge of Entertainment and Transportation, calls attention to Milwaukee's industrial variety; lists outstanding manufacturing plants which may be of interest to A. C. I. visitors at convention time.

The list follows. Please make note of any of these plants you would like to visit in Milwaukee. When you register at Convention headquarters, Shroeder Hotel, ask for forms on which to register for one or more trips to places of special interest. Trips for small groups will be scheduled so far as possible for Wednesday morning or afternoon, or Thursday morning, since the sessions in these periods appeal to special interests.

Mr. Smith's subcommittee is providing the transportation.

In addition to the factories and places of interest in the list the committee calls attention to the County Court House, now under construction, the Blue Mound Highway, constructed a year ago, the Electric Company's new rapid transit line for which considerable concrete has already been placed, besides the several ready-mix concrete plants in the city. Concrete products manufacturers visiting Milwaukee will not forget that this is aggressive concrete masonry territory where factories may be visited.

Make your wishes in these matters known to Mr. C. U. Smith or any member of his committee; someone will be on duty to represent him at the registration desk. The list of places of possible interest follows:

1. National Brake & Electric Co.; electrical control apparatus.
2. Seaman Body Corp., automobile bodies.

3. Worden - Allen Co., structural steel.
4. Lakeside Bridge & Steel Co., structural steel.
5. Wisconsin Bridge & Steel Co., structural steel.
6. A. O. Smith Corp., auto frames—pressed steel products.
7. Milwaukee Bridge Co., structural steel.
8. Koehring Machine Co., concrete mixers and power shovels.
9. T. L. Smith Co., concrete mixers.
10. Cutler-Hammer Mfg. Co., electrical controls.
11. Chain Belt Co., concrete mixers and conveying machinery.
12. Harnischfeger Corp., overhead cranes.
13. Kearney & Trecker, milling machines.
14. Allis-Chalmers Mfg. Co., general machinery, motors, etc.
15. Vilter Mfg. Co., refrigerating machinery.
16. Nordberg Mfg. Co., general machinery—Diesel engines.
17. Nash Motors Co., automobiles.
18. Bucyrus Erie Co., excavating machinery.
19. Milwaukee Sewerage Disposal Plant.
20. Milwaukee Municipal Harbor.
21. Lakeside Power Plant—T. M. E. R. & L. Co.

CAPPING CONCRETE BLOCKS FOR TEST

A LETTER FROM CHRISTOPHER BRANDA*

WE HAVE discussed recently with testing laboratories the method of capping our concrete blocks, in connection with tests made on behalf of our contractors.

These testing laboratories have been using plaster of Paris, for which, as authority, they have been able to quote Article 13 of the A. C. I. standard specifications, P-1A-29.

We believe, and our tests confirm our belief, that plaster of Paris flows out from surface of our blocks, and does not distribute the load uniformly. It so happens that the average wall thickness of our blocks is only $1\frac{3}{32}$ in., which undoubtedly exaggerates this condition. We believe, however, that the same condition exists in testing standard thickness building blocks.

The Lehigh Portland Cement Co., laboratory at Ormrod tested six blocks with plaster of Paris capping, and six with neat cement capping, obtaining about 30 per cent greater test values with neat cement.

At the time of the test, the neat cement caps were four days old; and the age of the neat cement cap and its curing also engage our consideration.

About a year ago, we read an article by P. J. Freeman, Chief Engineer, Bureau of Tests and Specifications of the Department of Public Works, Allegheny County, Pittsburgh, Pa., giving an account of capping with a mixture of sulphur and mineral fillers. It seemed to us that there would be many advantages in having a capping material that would harden quickly, and we accordingly have made some experiments with sulphur capping.

We have found that it is possible, and convenient to use sulphur alone, and that results with sulphur capping, and the character of failure indicate that full strengths were obtained. A test on a piece of sulphur, ¼ in. thick, 2 in. wide and 10 in. long gave a strength of over 8000 lbs. per square inch, and this was on a piece which was poured about a half hour before test.

Sulphur is inexpensive, and considering its great strength, the ease of capping, and the fact that blocks (or cylinders) could be tested immediately after capping, we strongly recommend sulphur as a standard capping medium.

*Republic Fireproofing Co., Inc., Bound Brook, N. J.

MEMBERS

FORTY ONE applicants for membership in December 1930 have been approved by the Board of Direction as follows:

Aldrich Construction Co., 111 W. Washington St., Chicago, Ill. (J. W. Trickel, Pres.)

American Sika Corp., 56 West 45th St., New York, N. Y. (Herbert Stoneham)

Ashland Vault Inc., 114 E. Seventh St., Ashland, Ohio. (H. A. Ledyard, Pres.)

Automatic Sealing Vault Co., The, 23 E. River St., Peru, Ind. (C. C. Millhouse, Treas. & Sales Mgr.)

Bates, Frederic G., 2310 Scottwood Ave., Toledo, Ohio.

Baumann, Alfred, c/o Scott & Urner Ltd., Caixa Postal 1863, Rio de Janeiro, Brazil.

Berg Vault Co., 1620 Lucas & Hunt Rd., St. Louis, Mo. (Christy L. Berg)

Branda, Christopher, 550 Winsor St., Bound Brook, N. J.

Carlson, Roy W., Box 93, Tujunga, Calif.

Congleton Co., W. T., Box 322, Lexington, Ky. (W. T. Congleton)

Crandell, John S., 308 Engineering Hall, University of Illinois, Urbana, Ill.

Eaton, E. C., 3440 Troy Drive, Los Angeles, Calif.

Foote Co. Inc., The, Nunda, N. Y. (Charles A. Lynch)

Hanneman Bros., 7841 South St., Detroit, Mich.

Hannock, Charles F., County Engineer's Office, Miami, Fla.

Hawley, Geo. W., 1100 N. St., Sacramento, Calif.

Howard, S. B., 26 Orchard St., Biddeford, Maine.

Knight, W. J., Pres., W. J. Knight & Co., 903 Warnerright Bldg., St. Louis, Mo.

Loizeaux Lumber Co., J. D., 861 South Ave., Plainfield, N. J. (George B. Coale)

Masonite Corporation, 111 W. Washington St., Chicago, Ill. (E. B. Bushnell)

Neff, Stewart S., 19 Arlington Pl., Buffalo, N. Y.

Norton, Paul W., 1330 Statler Bldg., Boston, Mass.

Ornelas, Samuel, Comision Nacional de Irrigacion, Est La Cruz, Chih, Mexico.

Painter, Geo. M., Belmont Cement Burial Case Co., West Manayunk, Pa.

Pasadena, City of, City Engineering Dept., 100 N. Garfield Ave., Pasadena, Calif.

Petursson, F., 123 Home St., Winnipeg, Man., Canada.

Phillips, Leroy, Box 150, Hubbell, Mich.

Pretoria Portland Cement Co. Ltd., P. O. Box 3811, Johannesburg, S. Africa. (W. F. Mackenzie)

Rusk, W. Harrell, c/o Raymond Concrete Pile Co., 140 Cedar St., New York, N. Y.

Ryan Ready Mixed Concrete Corp., 491 Smith St., Brooklyn, N. Y. (Joseph J. Ryan)

Schorer, Herman, 5925 Chabot Road, Oakland, Calif.

Seeliger, E., 43 Camp St., Cape Town, S. Africa.

Starrett Bros. & Eken, Inc., 101 Park Ave., New York, N. Y. (Russell H. Hunter)

Stuart, J. H., Stuart Vault Works, Bremen, Ohio.

Sureseal Manufacturing Co., 297 Ocean St., Jersey City, N. J. (Walter J. Bowman, Pres.)

Tripp, J. G., 8245 Idaho Ave., St. Louis, Mo.

Vaso Manufacturing Co., 519 Central Blvd., Seattle, Wash. (Carl L. Leon, Mgr.)

Weiler, Charles B., c/o Gifford Wood Co., Hudson, N. Y.

Willbee Concrete Products Co., 234 W. Euclid Ave., Jackson, Mich. (Charles E. Willbee)

Winnetka, Village of, Village Hall, Winnetka, Ill. (Robt. L. Anderson, Bldg. Insp.)

Wood, J. V., 110 Spencer St., Melbourne, Vict., Australia.

CAST STONE INSTITUTE

THE CAST STONE INSTITUTE will not meet as planned in Milwaukee Feb. 25. The organization's Executive committee has decided to postpone the annual meeting until spring or early summer.

INSTITUTE BOUND VOLUMES

THE INSTITUTE issues annual bound volumes of *Proceedings* on special order only. They are similar to the volumes 1 to 25 (1905-1929) except that the new volumes contain the *Abstracts*.

The *Proceedings* for any one year will comprise that part of each monthly *Journal* (September to June inclusive) containing papers, committee reports and discussions of them, with page numbers consecutive through the volume. The *Journal* is not published in July and August.

The *Abstracts* of current literature on cement and concrete, also published in the monthly *Journal* and given a separate series of consecutive page numbers through the year, will be bound with the *Proceedings*, gathered separately in the back of the book.

A member, or regular *Journal* subscriber may, through the Institute, have his own monthly pages of *Proceedings* and *Abstracts* sections from the *Journal* bound uniform with previous years, an advantage not always possible when the work is given to local binders. The cost for volume 26 (1930) was $1.75 to those who sent their *Journals* to the bindery at a specified time to get a quantity price. Later it was $2.10. Next year (for the volume concluded with the June 1931 *Journal*) the price may be somewhat higher—to be announced.

Members wishing to retain their monthly *Journals* separately instead of binding them, can obtain a bound volume (in addition to their monthly *Journals*) by ordering in advance of the issuance of the September JOURNAL each year, at a special member price of $3.00. The Institute will accept standing orders (from members only) for the 1932 and subsequent volumes at this additional charge of $3.00 until further notice. The regular member price for a bound volume when not ordered in advance is $6.00. That is the price now applying to Vol. 27 to be completed June 1931. The non-member price is $12.50 per volume.

Papers and Reports—In this Issue

Proceedings American Concrete Institute

Vol. 27—Pages 529-760

DESIGN AND CONTROL OF CONCRETE FOR DIABLO DAM..... 529
By H. F. FAULKNER and R. R. HUBBARD

SOME LONG TIME TESTS OF CONCRETE..... 547
By M. O. WITHEY

CINDERS AS CONCRETE AGGREGATE..... 583
Report of Committee 203, By EINAR CHRISTENSEN

ADMIXTURES AND WORKABILITY OF CONCRETE..... 647
By G. M. WILLIAMS

A COMPARISON OF CONTINUOUS WITH BATCH MIXERS IN PLANT OPERATION..... 655
By BENJAMIN WILK

DISCUSSION OF "BLAST FURNACE SLAG AS CONCRETE AGGREGATE"..... 661

DISCUSSION OF "CONSTRUCTION OF MAIN CANAL LINING ON KITTITAS DIVISION, YAKIMA RECLAMATION PROJECT, WASHINGTON"..... 669

REINFORCED CONCRETE COLUMN INVESTIGATION..... 675
Second Progress Report of Committee 105, W. A. SLATER, Chairman

Publication of discussion of the contributions in the following pages is on a time schedule. Please see inside cover pages.

Design and Control of Concrete for Diablo Dam*

BY H. F. FAULKNER† AND R. R. HUBBARD‡

DESIGN

THE DIABLO DAM is in a very rugged section of the Cascade Mountains on the Skagit River about 100 miles northeast of Seattle, Wash. It is the second major unit to be completed in the comprehensive development of that river to furnish hydro-electric power for the City of Seattle.

The first unit completed was the Gorge Creek tunnel and power plant, several miles down stream from Diablo. This consists of a 21-ft. diameter concrete lined tunnel driven 11,000 ft. through solid rock, terminating in three penstocks which lead to the three generators in the power house. A temporary crib dam and intake at the upper portal supplies water to this tunnel, which has a fall of 265 ft. in its 11,000 ft.

Diablo Dam is the first permanent dam to be constructed, and is the middle dam of three to be located on 14 miles of the river. Ruby Dam, much larger and higher than Diablo, will be next to be built, 7 miles up stream from Diablo; and later a permanent dam at Gorge Creek, 6 miles down stream, will complete the 1,000,000 H. P. development.

The base of Diablo Dam is in a narrow gorge which the river has cut through a dyke of hard granitic rock lying across the main valley floor. At the dam site the walls of this gorge rise almost perpendicularly to a height of 200 ft., above which elevation the slopes flatten. The average width of the gorge is 150 ft. at the dam site.

The central portion of the dam is of the constant angle arch type, terminating at each end in gravity abutment sections in

*For presentation and discussion 27th Annual Convention, Milwaukee, Feb. 24-26, 1931.
†Assistant Engineer, City of Seattle.
‡Superintendent of Construction, City of Seattle.

which are the spillways. The dam is 390 ft. high, with a crest length of 1170 ft.; has an extreme thickness of 140 ft. at the base and 16 ft. at the crest; and contains 320,000 cu. yds. of concrete. The arch portion of the dam contains 8 sections, each 73 ft. between contraction joints, having keys 6 ft. by 12 in. Three beaded copper water stops are placed vertically in each joint, connecting the adjacent sections. Two of these copper water stops are near the upstream face of the dam, on either side of a 6-in. block or prism of asphalt which was built vertically into the joint during construction and which was melted by electricity when the dam was completed, to effect a final sealing. The third copper water stop was placed near the down stream face of the dam. Perfection grout pipes were provided vertically in each key-way. These will be grouted after the dam has taken its final settlement under load.

At a large bend in the river about one-half mile below the dam site was an extensive sand and gravel deposit known as Reflector bar. Investigations disclosed that this bar would furnish an ample supply of aggregates for the dam and an excellent level site for the large construction camp of the contractors and engineers.

The concrete was designed and specified with particular recognition of the facts that the hydrostatic pressures would be very great, that permeability is in direct ratio to pressure, that any dissolving of the cement by percolating water is proportional to the quantity of percolating water, that it was necessary to obtain concrete of low permeability and that the arch type of dam, of relatively thin section, logically requires a stronger, more nearly impermeable concrete than does one of gravity section.

Samples of pit run material from various parts of Reflector bar were sent to the city laboratory at Seattle for preliminary studies. Tests indicated that there might be insufficient sand and a deficiency of finer particles in the sand available; that there would be plenty of intermediate gravel from ¼ in. to 3 in., with an excess of pea gravel, ¼ in. to ¾ in.; that there was plenty of large gravel and oversized boulders to supply any deficiencies in the aggregate if this larger material were crushed.

It was decided early not to use plums or very large aggregate, since we believe that such aggregate detracts from the work-

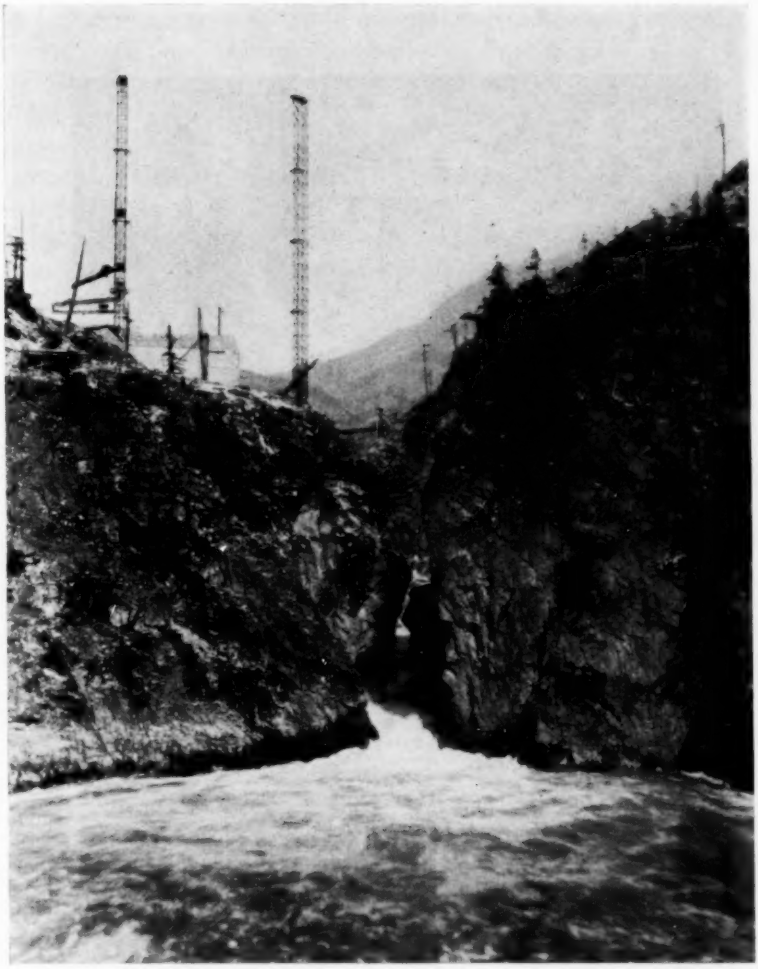


FIG. 1—GORGE OF SKAGIT AT DAM SITE

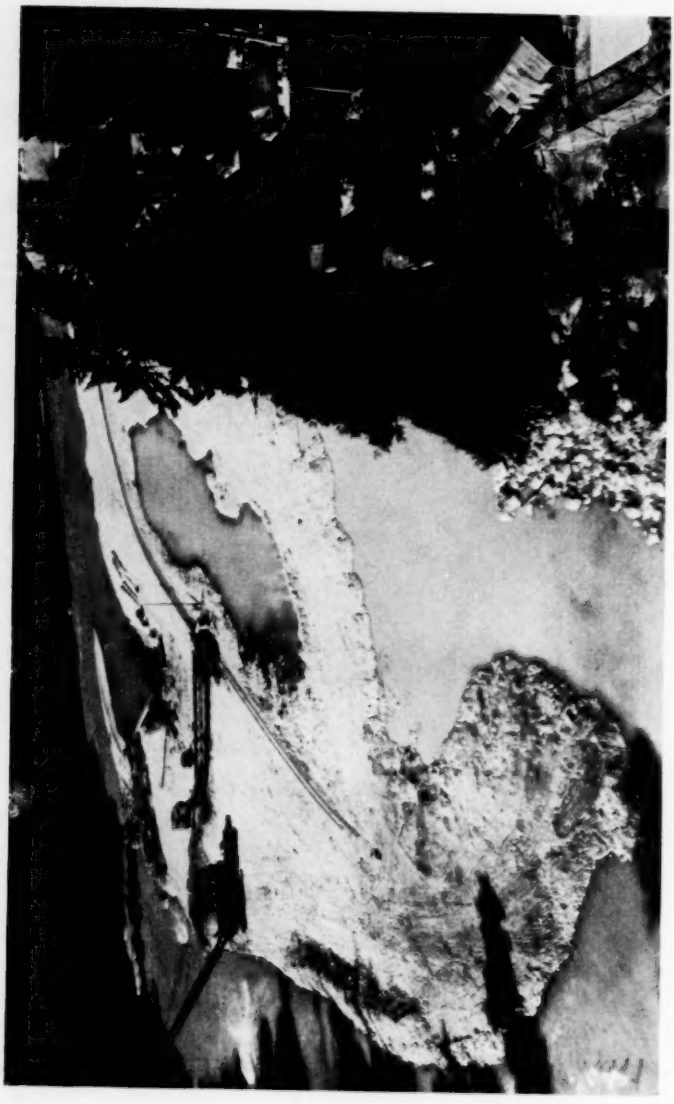


FIG. 2—REFLECTOR BAR—SOURCE OF AGGREGATE

ability or mobility of the fresh concrete mass and does not readily permit uniform placement and the impermeable structure necessary to secure watertight concrete. In the depositing and subsequent plastic settlement of concrete, water, the lightest ingredient, tends to rise. Coming in contact with the undersurface of large aggregate or plums, it can rise no farther but collects there as a film and through it the cement particles that may have been in contact with the aggregate surface, settle or disperse. Such water films create corresponding voids in the hardened concrete and afford ready passage to percolating water. In concrete of fairly stiff consistency such voids may be so thin as merely to cause a weakening of bond, but in wet concrete they may be so large as to be readily visible. It appears to be desirable to eliminate them.

For uniformity of mix it was decided to separate the coarse aggregate into two sizes, and to do all proportioning by weight. It was also decided to fix the maximum diameter of aggregate at 4 in., although permitting the use of elongated particles whose third dimension did in some cases run as high as 8 inches. After screening the samples and giving due consideration to the economic production of the bar, the following grading was specified:

Fine Aggregate: 0 to No. 4 with from 20 to 40 per cent passing a No. 28 sieve

Medium Aggregate:	Per cent passing 2″	95 to 100
	1½″	55 to 80
	1″	30 to 60
	½″	10 to 25
	¼″	0 to 5
Coarse Aggregate:	Per cent passing 4″	87 to 100
	3″	30 to 70
	2″	0 to 5

With the above grading of aggregates, the concrete mixture finally specified and used in the construction of the dam was determined by trial: various mixes containing various quantities of the different classes of aggregate were made and were compared as to their yield, strength, permeability, and (insofar as the elusive quality can be determined by both slump and by effort required to place properly in a test form) as to workability. Batches containing 20 lbs. of cement furnished sufficient con-

crete for the other tests and finally to make strength cylinders and permeability discs, which latter were 12 in. in diameter and 6 in. thick. The various aggregate proportions were tabulated in terms of their screen sizes and fineness moduli, but it was on the basis of the results obtained with the *trial mixes* that the final decision was made.

After making and testing specimens from the various trial mixtures, one using 20 lbs. cement to 160 lbs. of combined aggregate was decided upon as being most suitable. Making certain allowances for the rather coarse sand (F. M. 3.46) and the elongated shape of the coarse aggregate, and on the basis of the fineness modulus table, the selected mix had the following proportions, by weight:

1.0 parts cement
2.3 parts sand
3.5 parts medium aggregate
2.2 parts coarse aggregate

This mix produced a concrete which with a w/c of 0.75, 87.7 per cent solids and cement content of 1.19 barrels per cubic yard, could be readily placed, had a slump of 3½ in. when the aggregate over 2 in. in diameter was removed, and developed a compressive strength of 3,200 lbs. per sq. in. at 28 days. Best of all, the permeability discs, 6 in. thick, subjected to a pressure of 100 lbs. per sq. in. for 72 hours, remained dry on their unexposed faces for the entire period, such moisture as did penetrate through them being immediately removed by evaporation.

It was thought possible that the contractor might decide to crush the rock excavated from the dam foundations for use as aggregate. Since crushed aggregates do not produce as workable concrete for equal w/c, their use in the amount specified for gravel aggregate, i. e., 1.0 parts cement to 8.0 parts of graded aggregates, by weight, would require the use of more mixing water for equal plasticities, with a resulting loss of strength and increase of permeability. Therefore, it was specified that if crushed coarse aggregate were used, the mix should be 1.0 parts cement to 7.5 parts of combined aggregates, while if both fine and coarse

aggregates were crushed the mix should be 1.0 parts cement to 7.25 parts of combined aggregate.

The above is the "Class A" concrete which was specified for the main body of the dam.

In the upstream face of the dam to a depth of 3 ft. "Class B" concrete was placed integral with the "Class A" concrete. In this, the same gradation and relative proportion of aggregates was used, but the ratio of cement to aggregate was specified as one part cement to 5½ parts aggregate, by weight. This concrete required 1.60 barrels cement per cu. yd., and a w/c of .65 to produce the proper consistency. This richer mixture in the upstream face of the dam is an additional safeguard to a structure which is expected to serve for many centuries. The possibility of differential stresses between the two classes of concrete was considered, but it is believed that any such stresses will be minimized in that the gradation of the aggregate was the same in both classes of concrete.

Some of the principal requirements of the specifications were:

1. A mix of uniform fineness modulus, to be obtained by varying the quantities of the different aggregates.
2. Mixing time of 1½ minutes.
3. Concrete to be as dry as could be properly handled, but not to have more than 3-in. slump.
4. No chuting of concrete to be allowed.
5. No concrete to be dropped more than 3 ft.
6. Concrete to be spread over sections in layers not to exceed 6 in. thick.
7. Concrete not to be deposited in any section to a depth of more than 5 ft. at any one pour.

Some of these requirements were modified as the work progressed and as conditions appeared to warrant.

FIELD OPERATION AND CONTROL

A Telsmith screening and washing plant was installed by the contractor near the lower end of Reflector bar and river aggregate was used for the entire work. Sand and gravel were excavated by a Monoghan dredge and conveyed by a belt to the screening plant. This dredge could not furnish sufficient material at all times and a Sauerman drag line was installed to supplement the supply. The revolving screens at the plant had openings as follows for the production of the required sizes of aggregate:

For sand	⅛ in. by ¾ in.
For medium aggregate	2¼ in. square opening screens
For coarse aggregate	4½ in. square opening screens

Considerable difficulty was experienced at times in obtaining sand which contained the necessary amount of finer particles. The sand grading in the bar varied greatly in different locations. Usually if the dredge were working in coarse sand the Sauerman was placed where finer sand was known to be and vice versa.

Controlling the sand grading was something of a problem. Eight samples were taken at the screening plant from the stream of sand while it was filling a 38 yd. car in which the sand was conveyed from the bar to the storage bins at the damsite. These samples were mixed to form a composite sample, which was dried and screened. Certain tables and rules were worked out governing the acceptance of each car as follows:

If the percentage of sand retained on a No. 28 sieve was under 75, the fineness modulus could go as high as 3.60.

If the percentage retained was 76 to 80, the finess modulus must be 3.40 or under.

If the percentage was above 80, the car was rejected, or, as was the usual case, was held until the cars of finer sand were produced, which, on going into the storage bins at the same time, would bring the average grading within the specified limits.

Medium aggregate was comparatively easy to control. Pea gravel under ½ in. was almost entirely removed into a separate compartment at the plant. A sample from each car was screened and if the fineness modulus was found to be 7.90 to 8.00, 7 cu. yds. of pea gravel were added to the 38 yd. car:

If from 7.80 to 7.90	5.5 yards were added
If from 7.70 to 7.80	4.0 yards were added
If from 7.60 to 7.70	3.0 yards were added
If under 7.60 no pea gravel was added.	

There was no adjustment of the coarse aggregate except by the screens. All material brought to the plant between the 4½ in. and 8 in. sizes was put through a gyratory crusher, which was supposed to produce the 4 in. size, but for two reasons produced more medium than coarse material. These reasons were: first, the opening in the crusher was too small; second, the nature of the rock which, although having a compressive strength of 25,000 lbs. per sq. in. when tested in 3-in. cubes, was nevertheless

brittle, causing it to break into small pieces. The sizes produced with this crushed material, coupled with the plentiful supply of medium aggregate as it occurred in the natural deposit, developed an excess of the medium sized aggregate. Some of this excess was wasted, but much of it was stored and was either used for the reinforced sections of the dam or was sold to another contractor who used it for lining the tunnel between the dam and power house, for setting the penstock, and for building the surge chamber.

An average gradation of material produced is as follows:

	Fine Per Cent	Medium Per Cent	Coarse Per Cent
Ret. on 4″			12.8
3″			61.8
2″		4.1	91.1
1½″		20.0	95.2
1″		36.6	100
¾″		53.0	
⅜″		84.5	
No. 3	0.0	94	
4	2.5	100	
8	12.5		
14	41.5		
28	72.5		
48	93.5		
100	98.5		

The aggregates were loaded into 38 cu. yd. standard gauge cars and were taken to storage bins near the dam. These bins were cribs built on a steep hillside and held about 4,000 yards each of the three grades of aggregate, making the total storage 12,000 cu. yd. The material was drawn off through gates at the bottom of these cribs onto belts which conveyed it to small bins over the weighing hoppers at the mixing plant. Some difficulty was experienced with segregation of the coarse aggregate in the side hill bins, as, on drawing off, the coarser sizes would roll to the bottom, leaving the finer sizes on the slopes. This difficulty was overcome by putting back haul scrapers into the bins, which pulled the aggregate to the bottom with very little segregation.

Bulk cement was delivered to the job in 250 bbl. cars. This was scraped from the cars onto belts which conveyed it to silos

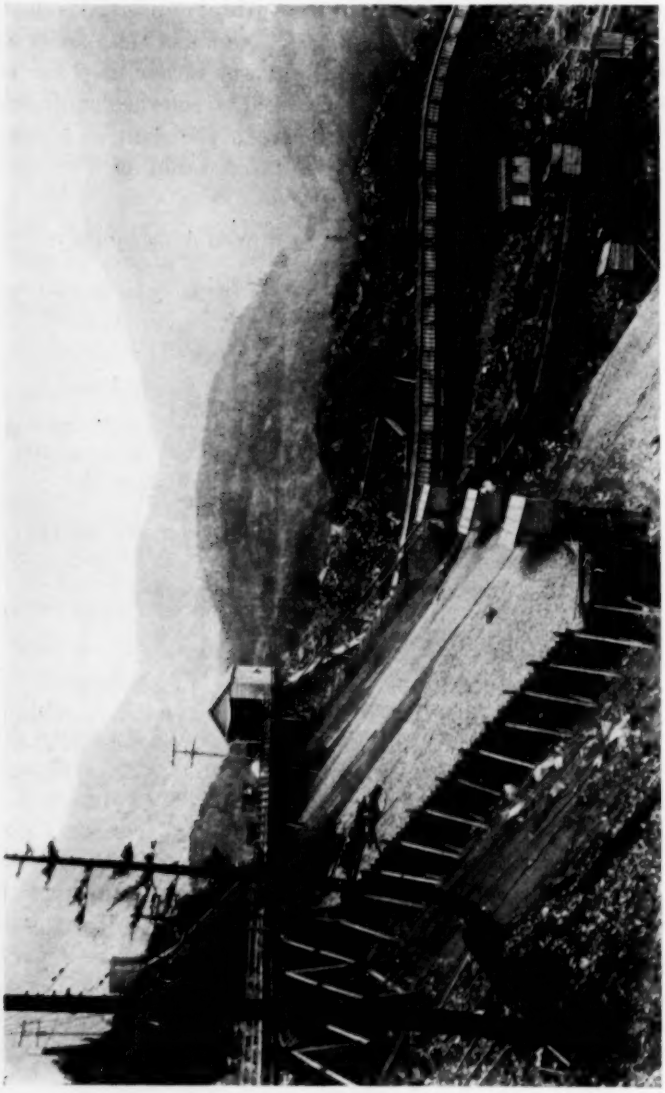


FIG. 3—AGGREGATE STORAGE AT DAM SITE AND BULK CEMENT CONVEYOR

at the mixing plant, from which a screw conveyor delivered it to the cement weighing hopper. Concrete was mixed by two 2-yd. Ransome mixers. Fine, medium and coarse aggregates were weighed in one hopper, three settings of the scale being necessary for each batch.

A typical mix for "Class A" concrete where none of the aggregate was crushed was: 940 lbs. cement, 2260 lbs. fine aggregate, 3142 medium aggregate, and 2090 coarse aggregate. Where a large percentage of both medium and coarse aggregate was crushed, as was frequently the case, a typical mix was 1003 lbs. cement, 2186 lbs. sand less 120 lbs. water, 3200 lbs. medium aggregate, and 2257 lbs. coarse aggregate. Moisture determinations of the sand were made and the entrained water was deducted from the quantity necessary for the proper consistency of the batch. A w/c of from .70-.75 was all that was required to produce a slump of 2½ to 3½ in. The water additional to that in the sand was measured very accurately by two 2-in. Neptune meters. These were accurate throughout the job to the nearest ¼ gal.

For the "Class B" or face concrete the aggregate mix remained the same, the only difference being that the cement content was increaesd to secure the ratio of 1 part cement to 5½ parts aggregate. From the mixing plant on the north rim of the gorge and about 100 feet downstream from the dam, the concrete was discharged into a hopper and thence was chuted down to the base of the dam, where two tall hoisting towers elevated it in skips to other hoppers on the towers located at the zone of construction operations. At the commencement of the concreting operations the concrete was discharged from the skips into 2-yd. bottom dump buckets which were handled by derricks. With the long cables necessary, this process was very slow and unsatisfactory, as it was difficult to position the heavy swinging buckets of concrete properly or to empty them without having the concrete pyramid and pile up in a manner requiring a great deal of shoveling and spreading to secure proper placement.

Later the contractors installed a belt conveyor system on the towers, which worked most efficiently. This installation con-

sisted of two 24-in. belts, the inner one being slung directly from the tower, while the outer one was suspended from a trolley on a steel boom. These belts were adjustable vertically on both tower and discharge ends, as the elevation of placement required. Each belt was driven independently by a small motor. Concrete discharged from the tower hopper onto the inner belt was conveyed outward and was discharged onto the outer belt through a short vertical pipe section which formed the hinge or elbow connecting the two belts. From this elbow the outer belt carried the concrete to its discharge into the "elephant trunk," suspended at the end of the outer belt. These belts were supported in a manner which allowed them to be jack-knifed through an arc of almost 360 deg. By proper jack-knifing of these belts a layer of the required thickness was spread rapidly and uniformly over the entire area of the section being placed, without segregation and requiring very little handling for placement. This conveyor handled as much as 1500 yds, in a 24-hour day. Our specifications required that concrete should be dropped not more than 3 ft., but it was found that better results could be obtained by dropping from 10 to 20 ft. with this "elephant trunk" method of depositing, and it was consequently permitted.

The concrete was of such consistency that ordinarily a man would sink about ankle deep in the fresh mass. During hot weather it was necessary to run the concrete somewhat wetter than a 3-in. slump, and during heavy rains which prevail at the damsite during the winter the consistency was reduced below a 3-in. slump, as water gains would occur as the pour progressed if not guarded against.

After a 5-ft. lift had been made, no more concrete was placed on that section until the concrete had hardened sufficiently to withstand the applied load without injury. Horizontal surfaces were prepared for succeeding pours by vigorous brushing with a stiff brush as soon as the concrete had stiffened sufficiently so that the sweepings were crumbly. The brushing removed the glaze and also any small amount of laitance which might have formed in low spots. Before fresh concrete was deposited the surface was picked where necessary, was hosed off under a high pressure water jet, and was then air blown. Upon this surface a

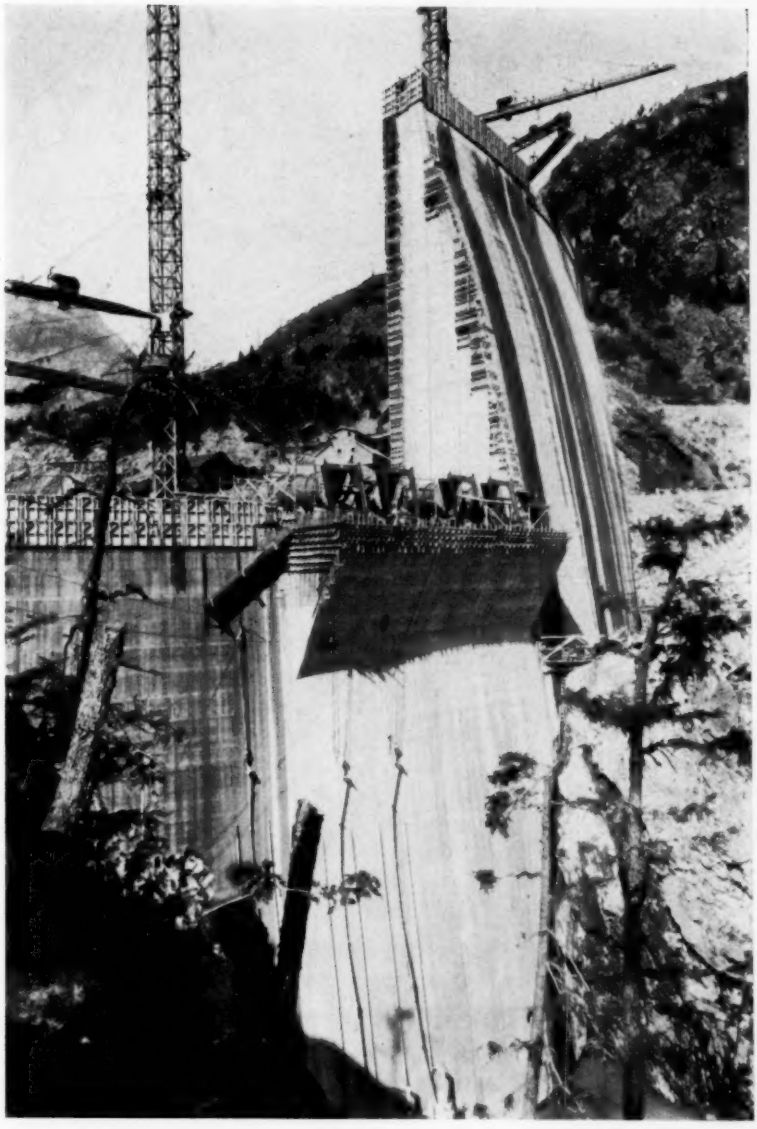


FIG. 4—UPSTREAM VIEW OF DAM DURING CONSTRUCTION

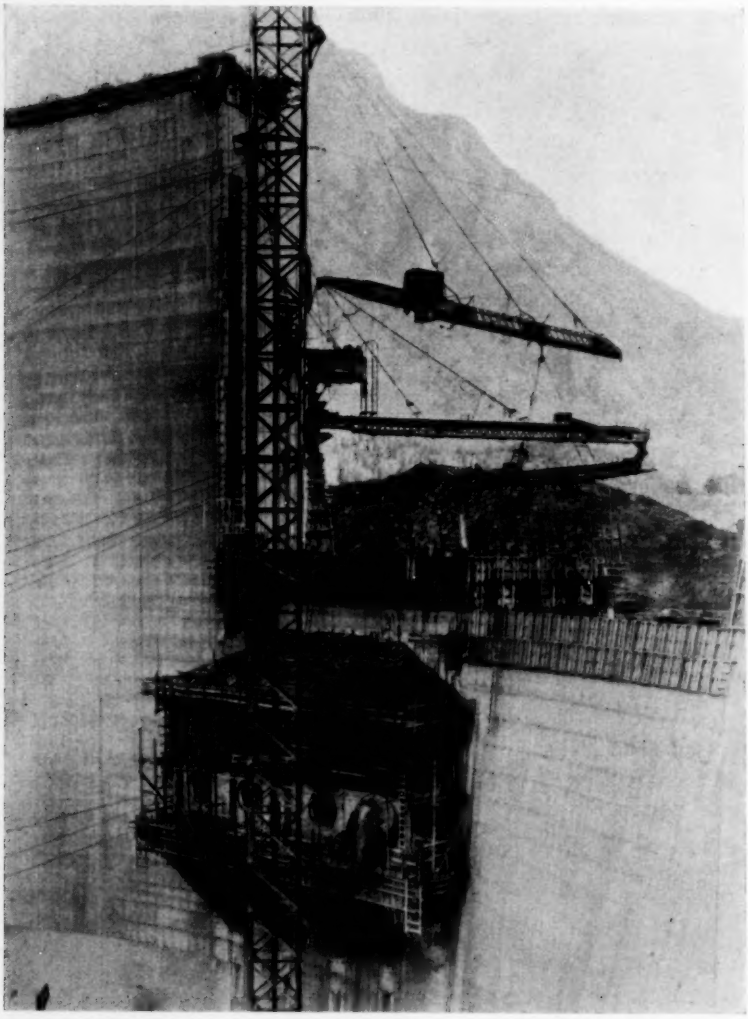


FIG. 5—DOWNSTREAM VIEW OF DAM UNDER CONSTRUCTION

layer of 1:1½ grout was thoroughly brushed in immediately preceding the concrete deposit.

On each 5-ft. lift a raised key 5 ft. wide and 15 in. high was placed 5 ft. back from the upstream face of the dam. About one foot back from the upstream face a strip of 26 gauge copper 8 in. wide was imbedded 4 in. in the surface of each 5-ft. lift, leaving 4 in. projecting to be imbedded in the next lift. This formed a water stop in the horizontal construction joints.

Concrete was cured by covering and sprinkling horizontal surfaces for 10 days, or until a fresh deposit was placed. The up and down stream faces of the dam were sprayed through perforated pipe hung along the top of the concrete. The ends of sections at the radial contraction joints were painted with asphalt as soon as the forms were removed.

No attempt was made to place concrete when the temperature was below 25 deg. F., as these occasions were rare and the plant was not provided with sufficient steam for protection in cold weather.

A few cracks developed in the dam during construction. They are entirely confined to the lower portion, where the sections are thick, and they occur on radial planes almost midway between contraction joints. None of these cracks has been traced through the dam, and they cease entirely at an elevation where the dam becomes about 40 ft. thick. Since the dam has been completed and filled no evidence has been observed of any leakage through the concrete itself. A few slight leaks have been found which come through the horizontal construction joints. Most of these are in proximity to the abutments and apparently come through cracks in the rock of the gorge wall, the water getting into the construction joints back of the keys and water stops. If keys and stops had been placed along the ends near the abutment walls, this condition would probably not have developed.

About 600 test cylinders of the various classes of concrete were made on this job. These were made by many different inspectors and were cured at the job laboratory in wet sand which was kept at a temperature of from 60 to 70 deg. F. until two days before test date, when they were packed in wet sawdust and shipped to the laboratory in Seattle. From the 6 by 12-in.

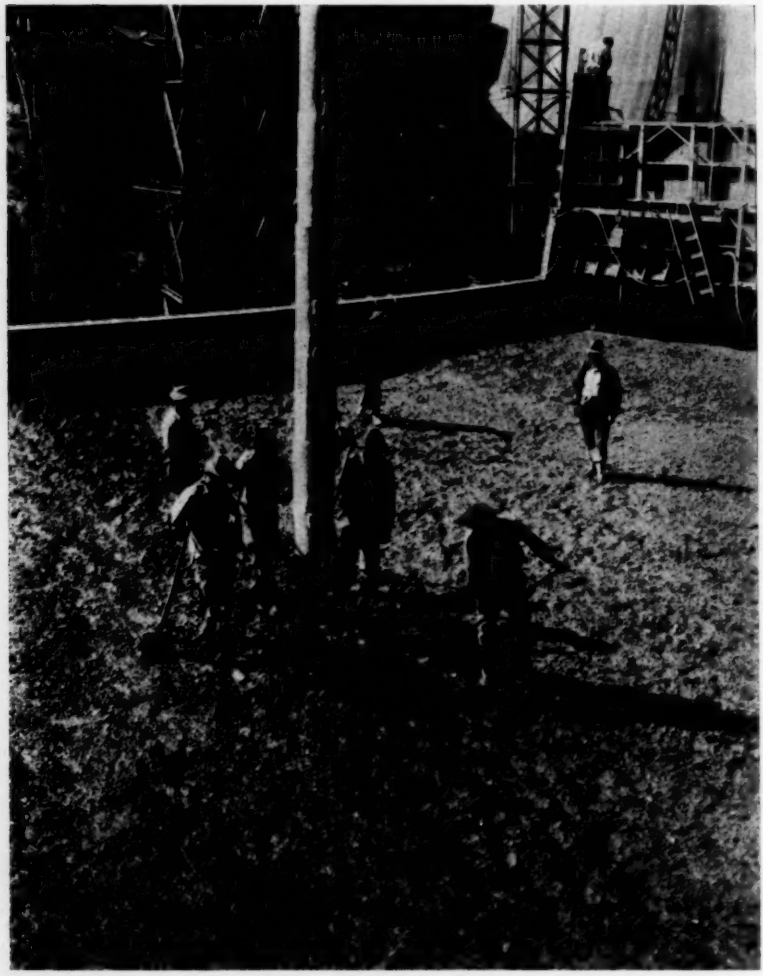


FIG. 6—CONCRETE BEING DEPOSITED THROUGH "ELEPHANT TRUNK"

cylinders aggregate coarser than 2 in. was removed; aggregate as large as 3 in. to 3½ in. was allowed in 8 by 16-in. cylinders. Of 350 "Class A" cylinders tested at 28 days:

2	failed under	2500 lbs. per sq. in.
11	failed between	2500 and 2800 lbs. per sq. in.
20	failed between	2800 and 3000 lbs. per sq. in.
53	failed between	3000 and 3500 lbs. per sq. in.
191	failed between	3500 and 5000 lbs. per sq. in.
52	failed between	5000 and 5500 lbs. per sq. in.
20	failed between	5500 and 6000 lbs. per sq. in.
1	failed over	6000

Average	7 day	"Class A" 6 x 12 in.	2662 lbs. per sq. in.
Average	28 day	"Class A" 6 x 12 in.	4297 lbs. per sq. in.
Average	6 mo.	"Class A" 6 x 12 in.	5709 lbs. per sq. in.
Average	1 yr.	"Class A" 6 x 12 in.	5907 lbs. per sq. in.
Average	28 day	"Class A" 8 x 16 in.	3817 lbs. per sq. in.
Average	7 day	"Class B" 6 x 12 in.	3377 lbs. per sq. in.
Average	28 day	"Class B" 6 x 12 in.	5100 lbs. per sq. in.
Average	6 mo.	"Class B" 6 x 12 in.	6350 lbs. per sq. in.
Average	1 yr.	"Class B" 6 x 12 in.	7250 lbs. per sq. in.

Permeability cylinders 8 by 9 in. were made from concrete as used at the dam and were taken to Prof. I. L. Collier at the University of Washington. These were subjected to 170 lbs. per sq. in. pressure and were found to be water tight.

The design and construction of this dam was under the direction of the City Engineer; T. H. Carver, Assistant City Engineer having direct charge. The Constant Angle Arch Dam Co., of San Francisco were consultants; E. D. Alexander was Resident Engineer; and Winston Bros., Minneapolis, Minn., Contractors.

Readers are referred to the JOURNAL *for June, 1931, for discussion which may develop. Such discussion should reach the Secretary by May 1, 1931.*

Some Long Time Tests of Concrete*

BY M. O. WITHEY†

INTRODUCTION

Two programs of experiments to ascertain the age-strength relationship for concrete cured under common conditions are in progress at The University of Wisconsin. The older test program, herein called Series A,[1] was begun in 1910 and includes crushing tests on 450 6 x 18-in. concrete cylinders of 1:2:4 and 1:3:6 proportions together with auxiliary tests on neat cement and standard sand mortars. Only one set of materials was used in these tests. The three curing conditions in Series A are: 1, storage in a water tank; 2, storage in a cage out of doors; and 3, storage in a cellar. Originally the program was planned to extend over a 10-year period but changes have been made in the test ages to extend the breaking periods to 50 years. Results of tests after 20 years storage are recorded in this report.

In the summer of 1923 a more comprehensive test program, Series B, was initiated. In this, four brands of cement, three coarse aggregates, three proportions, two consistencies, and two curing conditions were used. The total program included 1600 6 x 12-in. concrete cylinders for compression tests, 120 for absorption tests, and 30 for alternate freezing and thawing tests; 96 4 x 6 x 24-in. concrete slabs for expansion tests; 960 2 x 4-in. mortar cylinders, and 1080 briquettes. The curing conditions for this series are: 1. Out of doors as in Series A; 2. In a room in the laboratory.

It is planned to have these specimens tested at 10 intervals over a century. Results of tests up to 5 years are now reported.

*For presentation and discussion at 27th Annual Convention, Milwaukee, Feb. 24-26, 1931.

†Professor of Mechanics, College of Mechanics and Engineering, University of Wisconsin, Madison.

[1]For previous reports on this series of tests see *The Wisconsin Engineer*, Feb. 1915, Feb. 1918, and Nov. 1920.

Acknowledgments

The courtesy of the Wisconsin Granite Company, of Redgranite, in providing the crushed granite used in the tests of Series B is gratefully acknowledged.

Assistance in making the specimens of Series A was rendered by E. B. Nelson, '13; E. J. Paulus, '11; A. Ludberg, '11; and C. M. Scudder, '11. Messrs. S. D. Wonders, '13; C. M. Osterheld, '14; F. D. Bickel, '15; O. A. Bailey, '15; M. B. Hunder, '30; and L. M. Basford have assisted in testing and in preparation of data for reports.

The tests of Series B were made by C. A. Wiepking, with the assistance of R. S. Phillips, both at the time instructors in Mechanics, and W. Laflash, laboratory attendant. Mr. Wiepking also prepared many of the data for this report, A. H. Armstrong and M. B. Hunder assisted.

TESTS OF SERIES A

Materials

Cement—Atlas portland cement from Hannibal, Mo., was used in these tests. A sufficient supply for the entire program was mixed and stored in tightly covered milk cans. One 10-gallon can of this cement was set aside and retested from time to time when tests were made on the concrete specimens. Fig. 1 shows the effects of long storage in the tightly covered can on the tensile strength properties of this cement.

Aggregates—Janesville sand containing about 60 per cent quartz and 30 per cent dolomite was used in making the concrete specimens of the earlier tests. It weighed, when measured dry and loose, 109 lbs. per cu. ft. and contained 34.3 per cent voids. Approximately 97 per cent passed a ¼-in. sieve, 62 per cent passed a No. 30, and 3 per cent passed a No. 100 sieve. The specific gravity of the sand was 2.66.

A rather soft magnesian limestone crushed to pass a 1¼-in. sieve with 64 per cent passing a ¾-in., and 5 per cent passing a ¼-in. mesh was obtained from a local quarry and used in these tests. It weighed 87½ lb. per cu. ft. and contained 48 per cent voids. The specific gravity was 2.74.

Making and Testing Specimens

Making Specimens—The volumes of cement and air dry aggregate for each batch of 15 6 x 18-in. concrete cylinders were weighed; water was measured by a calibrated tank. Materials were mixed in a No. 0 Smith mixer ¾ min. dry and 2¼ min. after water was added. In emptying the mixer and in filling the molds care was taken to insure uniform disposition of the mixed concrete. Metal molds were assembled on leveled steel plates and every mold was filled about ⅓ full of concrete from each of the three wheelbarrows used in emptying the mixer. The concrete was puddled with an iron rod to expel air bubbles. After the concrete had partially hardened, tops of cylinders were capped with a thin layer of 1:1½ mortar.

Neat cement and mortar test pieces were made in accordance with the standard methods used in cement testing in 1910.

Curing—Concrete specimens were kept in molds three days, then marked, measured, weighed, and placed in a moist chamber where they were sprinkled twice a day for eleven days. Five specimens from each batch were subjected to each of the following curing conditions:

1. Cylinders were submerged in a 4½ x 4 x 2¾-ft. water tank at a temperature usually between 60 deg. and 70 deg. F. but occasionally dropping to 35 deg. F. for short periods in winter. During the first two months of storage the water in the tank was changed frequently, and for the first four years thereafter it was changed once a month. Subsequently it was changed at irregular intervals but usually three or four times a year.

2. Cylinders were placed with ends on the ground in an uncovered cage having a northeast exposure. The temperature range for the 20-year period was 25 deg. to 101 deg. F., the average was 46 deg. F. The mean humidity for that period was approximately 75 per cent with occasional departures of ± 10 per cent.

The average annual precipitation at Madison is 32 in. (includes snow). The average snowfal is 38 in. It is estimated that the specimens were subjected to 300 to 350 cycles of freezing and thawing during the twenty-year test period.

3. Cylinders were placed with ends in contact with the ground

in a cellar. The estimated range in temperature approximated 35 deg. to 70 deg. F. and the humidity range 50 to 75 per cent.

The neat cement and mortar specimens were stored one day in the moist closet and 13 days in a water bath at room temperature prior to being subjected to the previously mentioned storage conditions. In the cage and in the cellar these specimens rested on shelves.

One week before testing, concrete specimens were brought into the laboratory and were again weighed on the day broken. Neat cement and mortar specimens were tested immediately after removal from the given storage condition. In the early tests during the winter months frozen specimens were thawed before testing.

The schedule was so arranged that only one concrete specimen from a batch and curing condition was tested at a given time. For each age, mix, and curing condition five cylinders and three or four mortar specimens were broken.

Testing—Rough spots on the ends of cylinders and cubes were removed. In the tests made up to the ten year period two thicknesses of blotting paper were used on the top and bottom surfaces of compression specimens. In subsequent tests they were bedded with plaster of Paris. Speed of testing in the first seven years was somewhat less than 0.1 in. per min. Thereafter, the speeds were 0.06 in. per min.

Briquettes were tested in a Riehlé automatic shot briquette tester, but the roller bearings used in the early tests were inferior to those used in 1930.

Results of Tests

Tests on Stored Cement—The data in Fig. 1 and similar data on the compressive strengths of mortars show that portland cement may be stored for a very long time in a tightly closed vessel without suffering any material change in its strength properties. The changes represented by the data probably reflect discrepancies in testing procedure exercised by the different operators more than variability in properties due to age.

Effect of Curing Conditions on Strength of Mortars—The results of the tensile tests of neat cement and 1:3 standard sand mortar

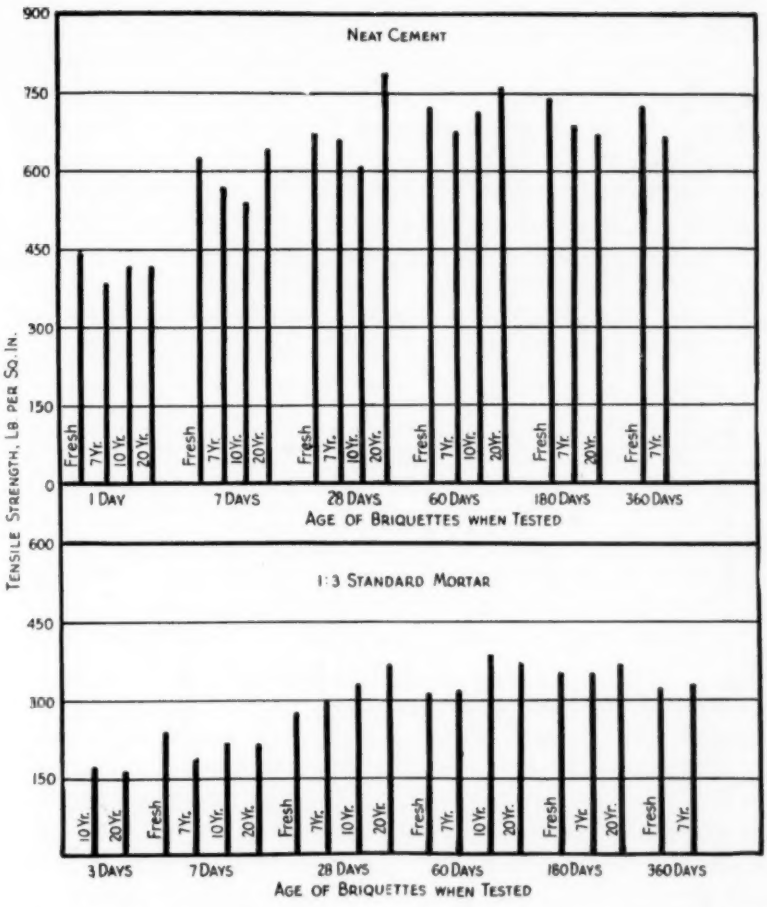


FIG. 1—THE EFFECT OF STORAGE IN A CLOSED MILK CAN ON THE TENSILE STRENGTH OF ATLAS PORTLAND CEMENT

briquets subjected to the different curing conditions are shown in Fig. 2. The neat cement specimens show wide variations in tensile strength under all three curing conditions. Much of this variability was undoubtedly due to surface cracks which were more or less pronounced in many of the neat briquets. The tensile strength of the 1:3 mortar briquets cured in the cage or

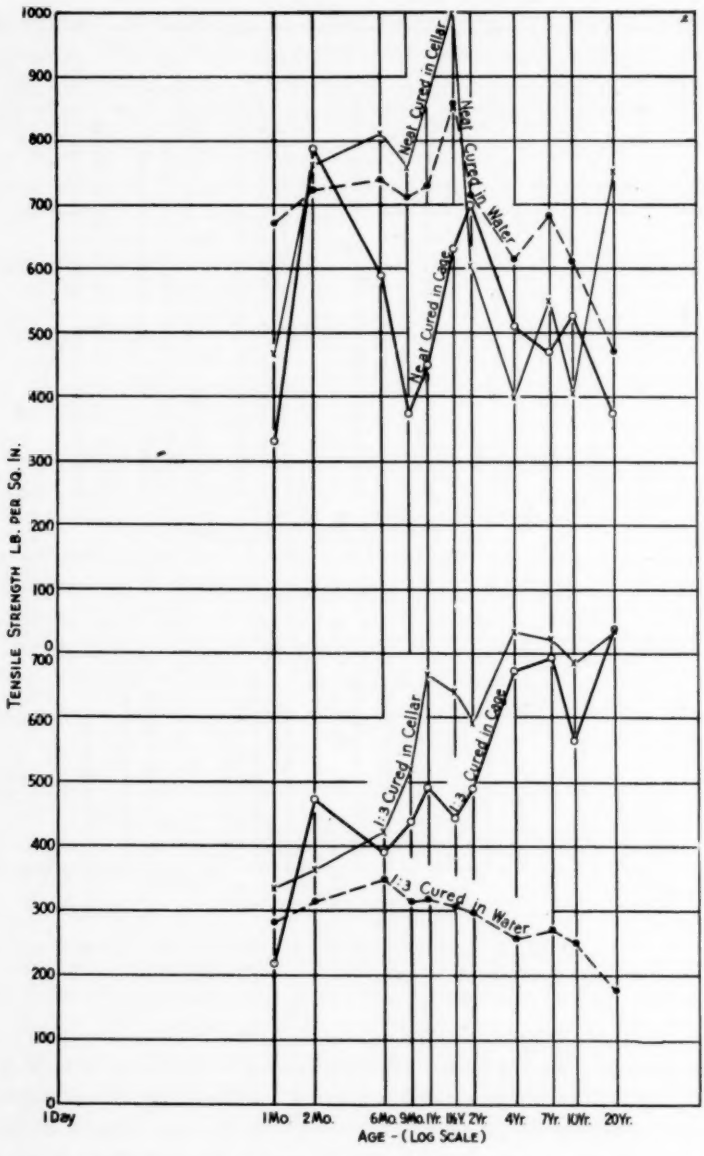


FIG. 2—RELATIONSHIP BETWEEN AGE AND CURING CONDITIONS ON THE TENSILE STRENGTH OF ATLAS CEMENT MORTAR BRIQUETS

in the cellar increased approximately as the logarithm of the age, as Fig. 2 shows, with the cellar-cured specimens showing slightly higher tenacity. On the other hand, the tensile strength of the water-cured 1:3 mortar specimens exhibited a decided retrogression after six months. Chemical analyses of the 1:3 briquettes subjected to the different storage conditions for 20 years showed the following cement contents in terms of the weight of the sand: for the cellar-cured specimens 34 per cent, for the cage-cured specimens 33 per cent, and for the water-cured specimens 31 per cent. From these data it seems likely that leaching out of the cement from the surfaces of the porous water-cured 1:3 briquets was the probable cause of their weakness in the late tests.

Effect of Curing Conditions on Compressive Strength of Concrete—Curves showing the relationship of the compressive strength of the concretes cured in the three ways to the logarithm of the age at test are shown in Fig. 3. Each point represents the average of five tests. It is apparent that the compressive strength of both the 1:2:4 and the 1:3:6 mixes cured out of doors or in the water tank increased approximately as the logarithm of the age. These two storage conditions gave approximately equal strengths. It is of interest to note that the 1:2:4 mixes cured under the more favorable conditions were slightly more than 2½ times as strong at 20 years as at one month, and that the 1:3:6 mixes under similar conditions were over 3½ times as strong. This increased percentage gain should not be interpreted as meaning that the weaker mixtures are gaining on the stronger, for Fig. 3 shows that the general trend of the curves is divergent and that the margin of strength between the rich and lean mixes is widening as the age increases. Both mixes of concrete cured in the cellar exhibited at 20 years about twice the one-month strength.

TESTS OF SERIES B

Materials

Cements—To ascertain the effect of different brands of cement on the age-strength relationships, three 10-bag shipments from each of four brands commonly used in Wisconsin were purchased from different consignments. Each shipment was sampled and tested and also the mixture of the three shipments of each brand. The four mixtures were used in the major tests of Series B. Table

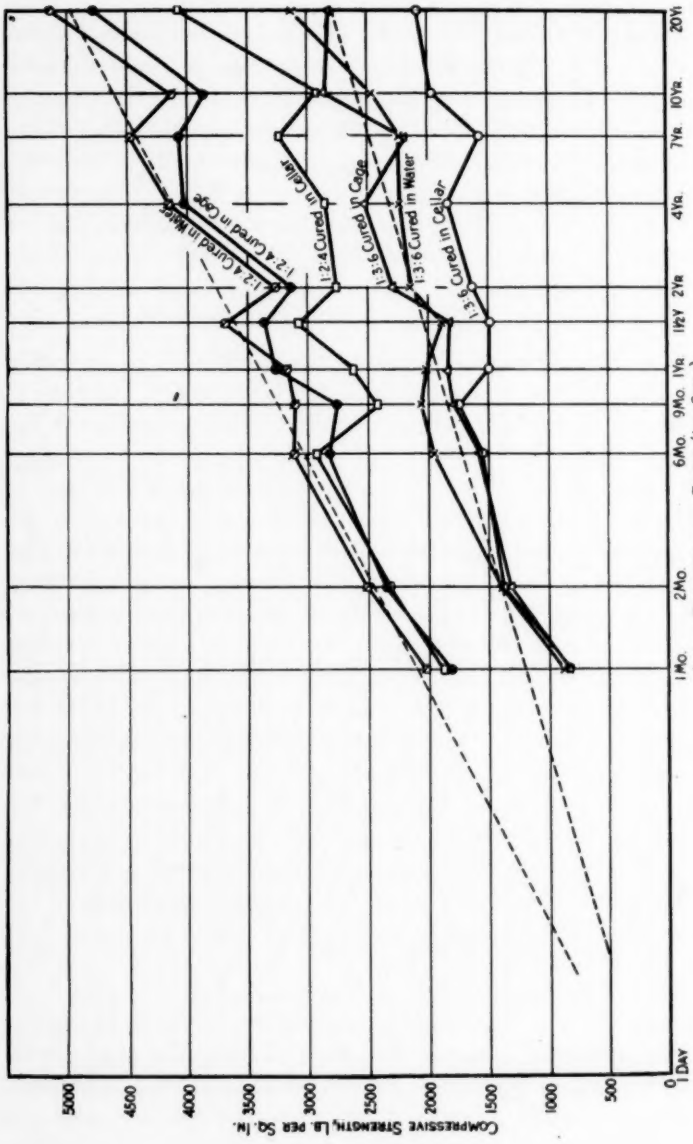


FIG. 3—STRENGTH-AGE CURVES FOR CONCRETE CURED IN VARIOUS MANNERS

1 shows results of standard physical tests on the mixtures representing the four brands. All brands exhibited good strength properties at 3, 7, and 28 days and passed other standard requirements. However, between the 90-day and 5-year periods all brands showed retrogression in both neat cement and mortar strengths. It is of interest to note that brand 3M which exhibited the greatest retrogression in mortar strength was the only brand which had a higher neat strength at five years than at one year.

TABLE 1—PHYSICAL PROPERTIES OF CEMENTS

Materials: Standard Ottawa sand. Cement samples taken from a mixture of 3 shipments.
All specimens were removed from molds at the age of one day and cured under water until tested.
Soundness in air, water, and steam, O. K.

Brand	Age		Tensile Strength, Lb. Per Sq. In.		Specific Gravity	Per Cent Water in Mix	Residue on Sieve No. 200	Time of Set					
								Initial				Final	
								Vicat		Gillmore		Vicat	Gillmore
	Years	Days	1:3 By Weight	Neat				Hr.	Min.	Hr.	Min.	Hr. Min.	Hr. Min.
Medusa (3M)		7	290	643	3.12	10.3	23.5	2	52	4	08	7 07*	7 53
		28	413	668									
		90	441	782									
	1	0	388	696									
	2	0	333	610									
	5	0	261	750									
Lehigh (4M)		7	279	608	3.14	10.2	21.7	2	12	3	32	5 25	6 52
		28	362	626									
		90	416	698									
	1	0	407	653									
	2	0	362	556									
	5	0	316	508									
Universal (5M)		7	264	667	3.13	10.3	20.7	1	55	3	12	5 00	6 28
		28	386	692									
		90	360	705									
	1	0	377	543									
	2	0	335	532									
	5	0	316	485									
Marquette (7M)		7	281	601	3.16	10.2	18.3	1	43	2	57	4 48	5 57
		28	394	643									
		90	427	672									
	1	0	437	592									
	2	0	390	514									
	5	0	372	486									

*Average of tests on individual shipments.

Chemical compositions of these cements are given in Table 2. The calculated constitutions of the cements are given in Table 9.

Autoclave tests were also made on the cements, but the results appear to have little correlation to the data thus far obtained in other tests.

TABLE 2—CHEMICAL ANALYSIS OF CEMENTS*

Brand	Specific Gravity	Chemical Analysis in Per Cent								
		SiO_2	CaO	MgO	Al_2O_3	Fe_2O_3	SO_3	Loss on Ignition	Total	Insoluble Residue
Medusa (3M)	3.12	22.53	61.40	4.39	5.43	2.56	1.73	2.32	100.36	0.22
Lehigh (4M)	3.14	22.21	61.16	2.57	7.32	2.69	1.90	1.64	99.49	0.44
Universal (5M)	3.13	21.41	62.55	2.30	7.18	2.57	1.53	2.40	99.94	0.16
Marquette (7M)	3.16	23.48	60.49	3.57	5.92	3.10	2.04	1.65	100.25	0.43

*Made by Professor O. A. Hougen.

TABLE 3—PROPERTIES OF AGGREGATES USED IN CONCRETE MIXES

Aggregate	Specific Weight, Lb. per cu. ft.	Specific Gravity	Per Cent Absorption	Fineness Modulus (m)	Sieve Analysis				
					Sieve No.	Per Cent Coarser Than Each Sieve			
						Janesville Sand	Janesville Gravel	Lannon Dolomite	Red Granite
Janesville Sand	112.6	2.70	0.60	2.86	1½	0	0	0	0
					¾	0	42.8	42.8	36.4
					⅜	0	85.7	85.7	90.9
Janesville Gravel	105.0	2.67	1.30	7.28	4	2.7	100.0	100.0	100.0
					8	17.5	100.0	100.0	100.0
					14	30.9	100.0	100.0	100.0
Lannon Dolomite	92.0	2.73	0.70	7.28	28	48.9	100.0	100.0	100.0
					48	88.1	100.0	100.0	100.0
					100	98.3	100.0	100.0	100.0
Red Granite	94.0	2.62	0.19	7.27		m = 2.86	m = 7.28	m = 7.28	m = 7.27

Janesville gravel contained approximately 50 per cent broken fragments and is composed of approximately ¾ dolomite, 1/5 quartz, the remainder being igneous material.

Janesville sand contained approximately 3/5 quartz, ¼ dolomite, and the remainder was largely igneous material.

Red granite is composed essentially of quartsite and feldspar.

Aggregates—Table 3 shows the physical properties of the aggregates used in tests of Series B. The Janesville sand and gravel are representative of the better types of glacial sand and gravel commonly used in concrete construction in Southern Wisconsin. Lannon dolomite and red granite represent hard and durable crushed rock aggregates of their respective types. The gravel particles were the most nearly equiaxed, the granite particles the least. It will be noted that all the coarse aggregates had approximately the same grading as indicated by the values of the fineness modulus (m).

Making and Testing Specimens

Auxiliary Test Specimens of Mortars—Besides the standard strength tests of cements, additional tensile and compressive tests of mortars made with standard sand and others made with Janesville sand are scheduled as outlined in Table 4.

TABLE 4—SCHEDULE OF MORTAR TESTS OF SERIES B

Brand of Cement	Kind of Sand	Mix by Weight	Curing Condition	Age at Test	Number of Specimens to be Tested			
					Per Mix Per Period		Total Made	
					2 x 4-in. Cylinders	Briquettes	2 x 4-in. Cylinders	Briquettes
3M 4M 5M 7M	—	Neat	Cage after 28 days in water	7 da. 28 da. 90 da. 1 yr. 2 yr. 5 yr. 10 yr. 25 yr. 50 yr. 100 yr.	3	3	120	120
As above	Standard	1:1 1:2 1:3	As above	As above	3	3	360	360
As above	Janesville	1:1 1:2 1:3	As above	As above	3	3	360	360
As above	Standard	1:3	In water	As above	3		120	
					Total.....		960	840

The consistencies for these mortar tests were based on Feret's formula,

$$w = \frac{2}{3} \frac{p}{s+1} + k;$$

where w = percentage of water for the given mix, s = number of parts sand to one of cement, by weight, p = percentage of water for standard neat consistency, and k = 6.5 for standard sand and 6.9 for Janesville sand.

Mortar briquets and cylinders were made in groups of six in separate batches. Specimens were numbered and stored so that not more than one specimen of a given batch was tested at an age. Specimens were cured one day in molds, 27 days in water. Those to be stored outdoors were then placed on wood shelves in the same cage used for the storage of specimens of Series A.

Concrete Test Specimens—Cylinders 6 in. in diameter and 12 in. high were made in batches of 20 in accordance with the program in Table 5. The materials were weighed and mixed in the same type of mixer as specimens of Series A. Dry cement and aggregate were mixed for ½ min., then the water was added and mixing was continued for 1½ min. Each batch from the mixer was discharged into wheelbarrows and thence onto a mixing tray where it was turned over to render it homogeneous. The requisite water for the desired consistency was determined by hand-mixing small preliminary batches. The consistency of every batch was checked on a 30-in. flow table using an 8 x 12 x 6-in. frustrum and 15 ½-in. drops in 15 sec. Molds were filled quarter full at a time and each layer was puddled 25 times with a ½-in. rod. A neat cement cap was placed on each cylinder about four hours after molding. After one day in the molds specimens were marked and stored for 27 days in a moist closet. Then they were removed and subjected to the appropriate storage conditions. The outside curing is conducted in the same cage as the specimens of Series A. The inside curing is done in a room where the temperature ranges from 65 to 85 deg. F. and the humidity ranges from 40 to 80 per cent. The schedule is so arranged that not more than two cylinders from a given batch are broken at a given age.

The 4 x 6 x 24-in. contraction prisms were made from certain of the batches used in making cylinders. Brass plugs spaced

TABLE 5—SCHEDULE OF CONCRETE TESTS OF SERIES B

Janesville sand was used in making all specimens.
Specimens for compression, absorption, and freezing tests were 6 x 12-in. cylinders; for contraction tests were 4 x 6 x 24-in. prisms.

Type of Test	Brand of Cement	Mix by Vol.	Coarse Aggregate	Consistency*	Flow	Curing Conditions	Age at Test	Number of Specimens to be Tested			
								Per Mix Per Period		Total Made	
								Cyl.	Prisms	Cyl.	Prisms
Compression and Expansion	3M 4M 5M 7M	1:2:4	Janesville Gl. Lannon Dol. Red Granite	110	165	Cage	7 da. 28 da. 90 da. 1 yr. 2 yr. 5 yr. 10 yr. 25 yr. 50 yr. 100 yr.	5	3	600	36
Compression and Contraction	As above	1:1½:3	Janesville Gravel	110	165	Cage		5	3	200	12
		1:3:6		110	165	Cage	As	5	3	200	12
		1:2:4		140	250	Cage	above	5	3	200	12
		1:2:4		110	165	Indoors		5	3	200	12
		1:2:4		140	250	Indoors		5	3	200	12
Absorption and Freezing	As above	1:2:4	Janesville Gl.	110	165	Moist clos.	28 da.	10		40	
		1:2:4	Lannon Dol.	110	165	Moist clos.	28 da.	10		40	
		1:2:4	Red Granite	110	165	Moist clos.	28 da.	10		40	
		1:1½:3	Janesville Gl.	110	165	Moist clos.	28 da.	10		40	
		1:3:6	Janesville Gl.	110	165	Moist clos.	28 da.	10		40	
		1:2:4	Janesville Gl.	140	250	Moist clos.	28 da.	10		40	
										1840	96

*Percentage of water used in mix based on net amount required for a slump of one inch.

TABLE 6—AVERAGE CEMENT CONTENTS, WATER CEMENT RATIOS AND DENSITIES OF MIXES

Mix By		Coarse Aggregate	Cement Content Sacks Per Cu. Yd.	Water Cement Ratio, w/c	Flow in Per Cent	Specific Weight, Lb. Per Cu. Ft.	Density *p*
Vol.	Weight						
1:2:4	1:2.4:4.47	Gravel	5.15	0.76	165	151.5	0.828
1:2:4	1:2.4:4.47	Gravel	5.04	1.01	250	150.7	0.808
1:2:4	1:2.4:3.92	Lannon Dol.	5.52	0.81	165	151.6	0.814
1:2:4	1:2.4:4.00	Red Gr.	5.41	0.77	165	149.1	0.824
1:3:6	1:3.6:6.70	Gravel	3.58	1.04	165	150.5	0.830
1:1½:3	1:1.8:3.35	Gravel	6.62	0.61	165	152.2	0.828

20 in. apart were molded in the axis of each 6 in. face. These were provided with holes for strain gage readings. The slabs were subjected to the same curing conditions as the cylinders.

The 6 x 12-in. cylinders for the absorption and freezing tests were selected from those made in different batches. The absorption cylinders were roughened with wire brushes when one day old. For both types of test the cylinders were cured in the moist closet for 27 days.

Methods of Testing

Absorption Tests—After the roughened 6 x 12-in. cylinders for the absorption tests had been stored 27 days in the moist closet they were dried to constant weight at temperatures just above the boiling point of water. They were then immersed in water at room temperature for 48 hr. to determine the percentage of absorption. After an interval of a few days the cylinders were boiled in water for 5 hr. to ascertain how much additional water they would absorb. The percentages of absorption were based on weights.

Method for Freezing and Thawing Tests—The cylinders to be subjected to the freezing and thawing tests, after curing in moist closet for 28 days, were immersed in hot water, at approximately 190 deg. F., for several hours to saturate them. They were cooled in running tap water and then placed in a freezing chamber having an average temperature of 21 deg. F., and kept there until thoroughly frozen. All of these specimens were subjected to twenty such cycles. They were then broken in compression.

Contraction Measurements—Measurements on the lengths of the 4 x 6 x 24-in. contraction prisms were made with a 20-in. Berry strain gage reading to 0.0001 in. Initial readings were taken when the specimens were one day old. They were again read after 27 days in the moist closet while still in the closet and then removed to the assigned storage condition. Measurements of lengths and weights have since been taken at regular intervals, usually at the test periods for the concrete cylinders.

Procedure in Strength Tests—Mortar and concrete specimens stored in the cage have been removed to the laboratory and stored two days prior to testing. Other specimens were carried directly from storage to testing machine. Wherever necessary

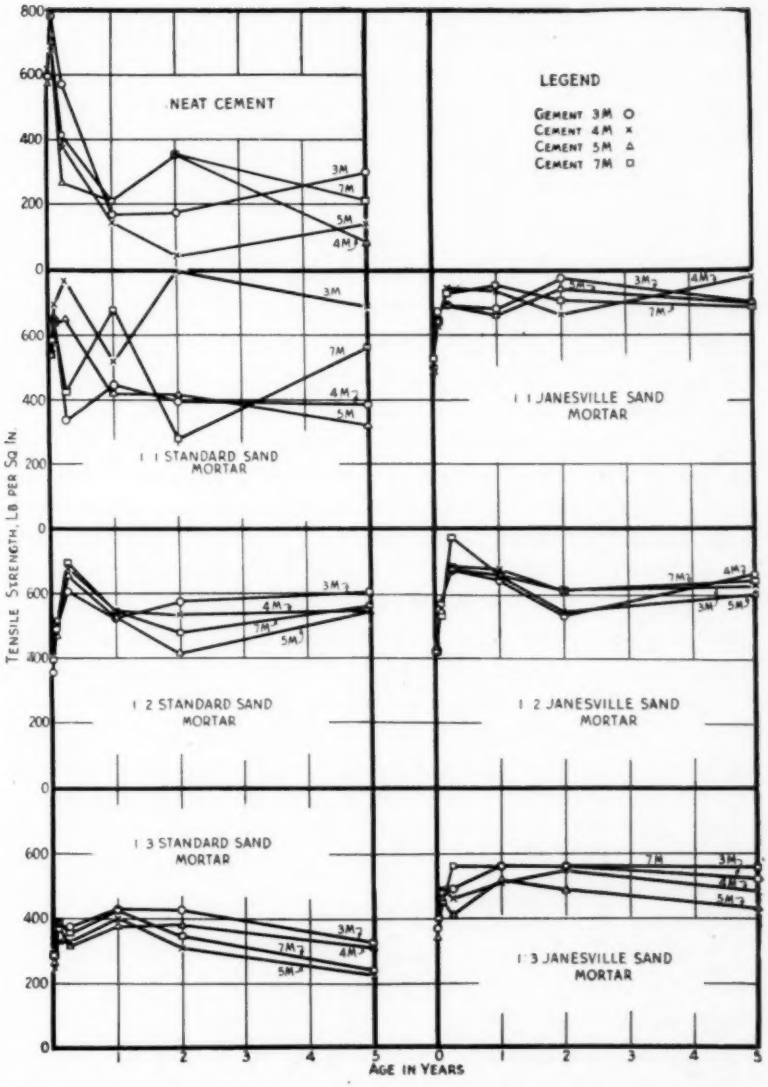


FIG. 4—RESULTS OF TENSION TESTS ON MORTAR AND NEAT CEMENT BRIQUETS CURED OUTSIDE AFTER 28 DAYS

defective caps have been removed from compression cylinders and they have been recapped with plaster of Paris.

The 6 x 12-in. concrete compression cylinders have been broken on a 200,000-lb. 3-screw Riehlé testing machine equipped with a spherical bearing on the movable head. The speed used in applying maximum loads was 0.06 in. per min.

The 2 x 4-in. mortar cylinders have been broken at a speed of 0.03 in. per min. under a small spherical bearing.

Briquettes have been tested in a Riehlé automatic shot briquet tester equipped with standard roller bearings.

RESULTS OF TESTS OF SERIES B

Mortar Strength Tests—Fig. 4 shows the effect of outside curing on the tensile strength of briquets made in 1923. These data show that neat cement briquets and 1:1 standard sand mortar briquets suffered severely from weathering. In some instances briquets of these rich mixes were badly checked and gave little strength. Since these very low values were averaged with the other results, they account for the marked fluctuations in the tensile-strength-age curves. Why the tenacity of the 1:1 mortars of standard sand suffered so markedly in contrast to 1:1 mortars of Janesville sand is not apparent. All 1:3 standard-sand mortars cured outside exhibited a pronounced retrogression in tensile strength after the one-year period. The 1:3 Janesville-sand mortars also showed a slight retrogression after the one-year period. The 1:2 mortars and the 1:1 Janesville-sand mortars showed little change in tensile strength after the one-year period.

The results of the compression tests (not presented herein) on 2 x 4-in. mortar cylinders cured outside show much less variation in compressive strength than in tensile strength. Furthermore, there is no pronounced retrogression in strength with age for any of the mixes. As in the tensile tests, the Janesville sand mortars exhibited compressive strengths superior to those attained by standard sand mortars, especially in the 1:3 mixes.

In the mortar tests the influence of brand on strength was not large. Considering the results of tests with Janesville sand and averaging the data for the 28-day to 5-year periods, brand 7M showed the highest tensile strength and brand 5M the lowest. The difference between the average for the highest brand and the

lowest brand amounted to 4 per cent for the 1:1, 7 per cent for the 1:2, and 15 per cent for the 1:3 mix. In compression brand 4M gave the highest average strength and brand 5M the lowest. The difference between the average for the highest and lowest brands was 9 per cent for the 1:1 mix, 8 per cent for the 1:2 mix, and 6 per cent for the 1:3 mix.

The differences in tensile strengths of the neat cement mixes due to curing conditions was very marked. Neat briquettes stored in water exhibited much higher and more uniform strengths than those stored outside. The discrepancies in the tensile strengths of the 1:3 standard sand mortar briquets due to the two storage conditions was not marked, but the 1:3 cylinders of standard sand cured outside averaged about fifty to seventy-five per cent stronger than companion specimens cured in water at room temperature.

It is believed that the wide discrepancies in the tensile strengths of the specimens cured outside was due to surface checks which seriously diminish tenacity, especially of small specimens, but which affect the compressive strength little.

Physical Properties of Concretes—Data on average cement contents, water-cement ratios, consistencies and densities of the concretes used in these tests are given in Table 6. The water-cement ratios are based on ratio of the volume of free water (absorbed water subtracted) to the volume of cement. Cement contents, specific weights, and densities were determined on concrete after removal from molds.

Results of Absorption Tests—Data on the absorption tests are given in Table 7 and in Fig. 5. The absorptions of the individual cylinders were very uniform in most cases, the average variation from the mean in a group of five was less than 5 per cent and in only one case did it reach 8 per cent. The data in column 7 of Table 7 show the variations of the mean for each brand from the average for the given mix.

The absorption for the 1:2:4 Red granite mix was least, and that mix also had the lowest ratio of absorption after 4-hr. immersion to absorption after boiling. Considering tests with Janesville gravel, the absorption appears to increase as the water-

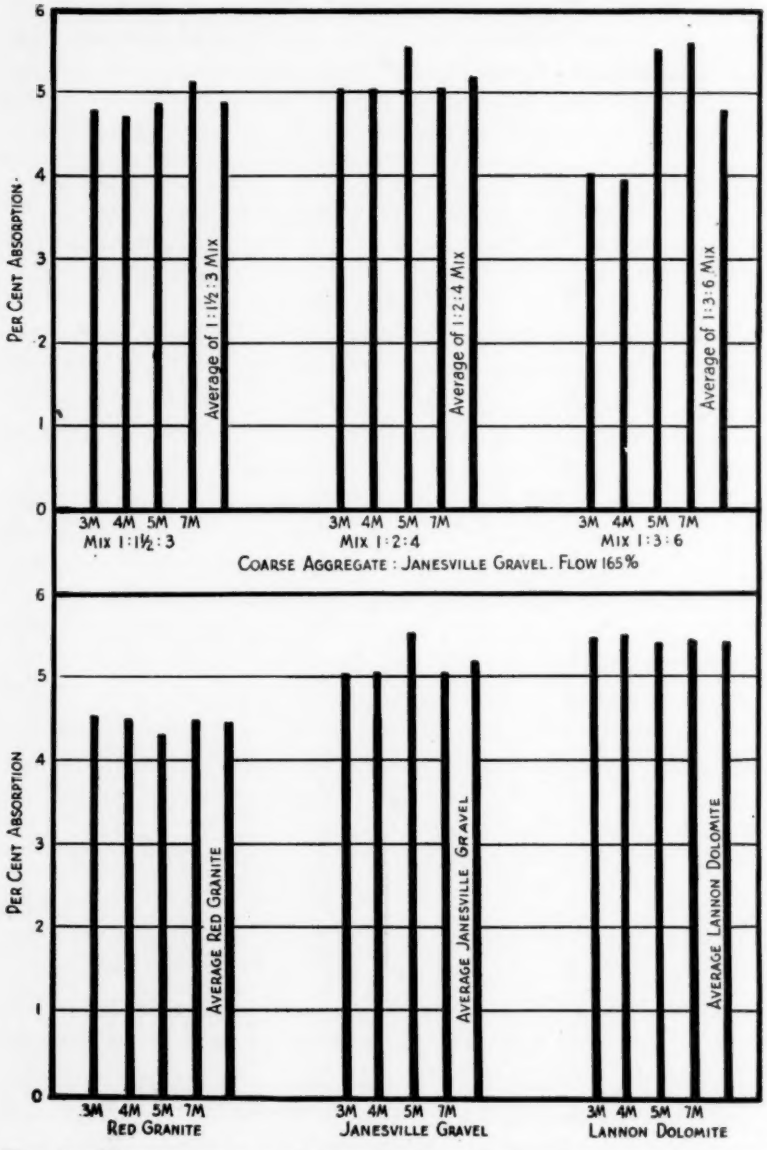


FIG. 5—EFFECT OF MIX AND TYPE OF AGGREGATE ON ABSORPTION OF CONCRETE

TABLE 7—RESULTS OF ABSORPTION TESTS ON CONCRETE

Specimens were stored in moist closet for 27 days, dried to constant weight, then immersed in water for 48 hours. After a few days the cylinders were boiled in water for half a day to show how much more additional water would be absorbed by this treatment.

Each value represents the average of 5 test specimens.

Brand of Cement	Mix	Flow, Per Cent	W/C Ratio	Aggregate	Per Cent Absorption at 48 Hr. (A)	Per Cent Variation from Ave.	Per Cent Increase in Weight Due to Boiling 5 Hours in Water After Drying (B)	Ratio, A/B
3M	1:2:4	165	.81	Lannon	5.51	+ 0.7	6.36	.87
4M			.81	Dolomite	5.47	0.0	5.84	.93
5M			.81		5.44	— 0.6	6.06	.90
7M			.81		5.47	0.0	6.20	.88
Ave.					5.47		6.11	.89
3M	1:2:4	165	.77	Red	4.59	+ 2.2	5.44	.84
4M			.77	Granite	4.53	+ 0.9	5.49	.82
5M			.77		4.34	— 3.3	5.19	.84
7M			.77		4.52	+ 0.7	5.37	.84
Ave.					4.49		5.37	.84
3M	1:2:4	165	.76	Janesville	5.02	— 2.9	6.14	.82
4M			.76	Gravel	5.03[a]	— 2.2	5.62[a]	.90[a]
5M			.76		5.58	+ 7.9	6.32	.88
7M			.76		5.04	— 2.5	5.77	.87
Ave.					5.17		5.96	.87
3M	1:2:4	250	1.01	Janesville	5.85	— 3.8	6.58	.89
4M			1.01	Gravel	6.30	+ 3.6	7.02	.90
5M			1.01		6.55	+ 7.7	7.16	.91
7M			1.01		5.64	— 7.2	6.64	.85
Ave.					6.08		6.85	.89
3M	1:1½:3	165	.61	Janesville	4.78	— 1.8	6.01	.80
4M			.61	Gravel	4.71	— 3.3	5.71	.82
5M			.61		4.83	— 0.8	5.68	.85
7M			.61		5.17	+ 6.2	5.73	.90
Ave.					4.87		5.78	.84
3M	1:3:6	165	1.04	Janesville	4.02	—16.1	4.66	.86[a]
4M			1.04	Gravel	3.95	—17.5	4.31	.92[b]
5M			1.04		5.55	+15.9	6.19	.90
7M			1.04		5.65	+18.0	6.35	.90
Ave.					4.79		5.38	.90

[a]This value represents the average of 4 test specimens.
[b]This value represents the average of 2 test specimens

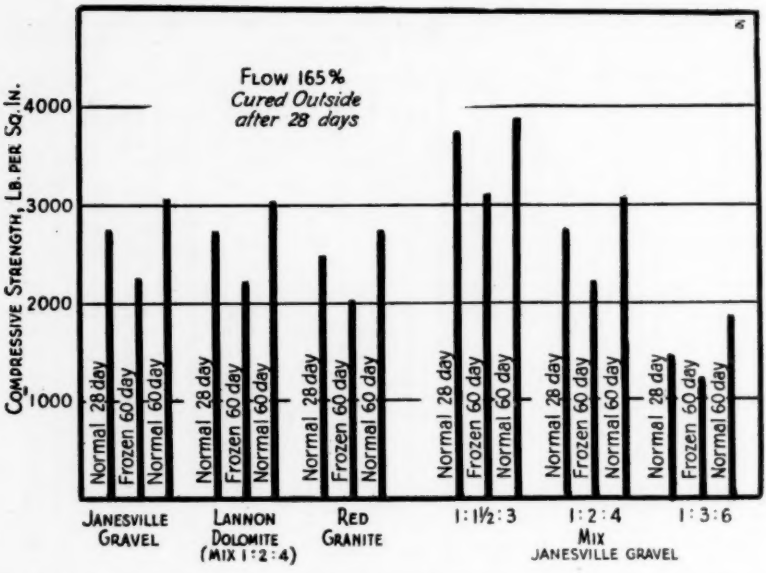


FIG. 6—EFFECT OF AGGREGATES AND PROPORTIONS ON STRENGTH OF CONCRETE SUBJECTED TO FREEZING AND THAWING

cement ratio (w/c) increases. A higher w/c undoubtedly accounts for the absorption of the 1:2:4 Lannon stone mix being greater than the absorption of the 1:2:4 mix of Janesville gravel.

Neglecting values for the 1:3:6 mix, in which the values for brands 3M and 4M appear abnormal, the average absorptions for 25 specimens of a given brand were:

Brand	3M	4M	5M	7M
Average Percentage Absorption	5.15	5.21	5.35	5.17

Hence there appears to be little difference in the absorption of concrete made from these brands of cement.

Results of Freezing and Thawing Tests—The data from the freezing and thawing tests on the concrete cylinders are summarized in Table 8 and Fig. 6. Although the freezing and thawing tests were started when the specimens were 28 days old the

TABLE 8—RESULTS OF FREEZING TESTS

Test specimens were cured normally for 28 days, then subjected to 20 cycles of freezing at a temperature of approximately 21° F. and thawing in water heated at a temperature of approximately 190° F. Temperature reversals extended over a period of one month.

All normal specimens were cured outside after 28 days.

The 60-day normal strength values were interpolated from strength curves plotted against the logarithm of the age.

A/B is the ratio of the percentage absorption at 48 hours to the percentage increase in weight due to boiling 5 hours in water after drying.

Mix	Coarse Aggregate	Cement Brand	W/C Ratio	Flow, Per Cent	Comp. Strength, Lb. per Sq. In.				Ratio, A/B
					Concrete Subjected to Freezing, Age 60 Days	Normal Concrete, Age 90 Days	Normal Concrete Corresponding to Age of Frozen	% Reduction in Str. of Frozen Concrete Based on 60-Day Normal Str.	
1:2:4	Granite	3M	.77	165	1960	2465	2285	14	.87
		4M	.77	165	1995	3065	2845	30	.93
		5M	.77	165	2060	2985	2890	29	.90
		7M	.77	165	2265	3155	3075	26	.88
Ave.					2070	2915	2775	25	.89
1:2:4	Lannon Dolomite	3M	.81	165	2150	2820	2705	21	.84
		4M	.81	165	2565	3260	3085	17	.82
		5M	.81	165	1995	3310	3050	34	.84
		7M	.81	165	2140	3400	3360	36	.84
Ave.					2215	3200	3050	27	.84
1:2:4	Janesville Gravel	3M	.76	165	2250	3110	2875	22	.82
		4M	.76	165	2285	3695	3435	33	.90
		5M	.76	165	1860	2785	2840	34	.88
		7M	.76	165	2640	3325	3145	16	.87
Ave.					2260	3230	3075	27	.90
1:2:4	Janesville Gravel	3M	1.01	250	1255	1985	1800	30	.89
		4M	1.01	250	1535	2470	2230	31	.90
		5M	1.01	250	1345	2130	2130	37	.91
		7M	1.01	250	1655	2440	2220	25	.85
Ave.					1445	2255	2095	31	.89
1:1½:3	Janesville Gravel	3M	.61	165	2795	3690	3560	21	.80
		4M	.61	165	3300	3635	3575	8	.82
		5M	.61	165	3275	4230	4165	21	.85
		7M	.61	165	3365	4480	4345	23	.90
Ave.					3185	4010	3910	19	.84
1:3:6	Janesville Gravel	3M	1.04	165	1100	1610	1520	28	.86
		4M	1.04	165	1140	2235	1975	42	.92
		5M	1.04	165	1400	2105	1880	25	.90
		7M	1.04	165	1285	2205	1980	35	.90
Ave.					1230	2040	1835	33	.90

20 cycles were not completed until the specimens were about 60 days old. Since no tests were made on normal concrete at 60 days, for purposes of comparison the strengths of normal specimens in column 8 of Table 8 were estimated by interpolating between the 28-day strengths and the 90-day strengths. In making this interpolation the strength was assumed to vary with the logarithm of the age.

It will be noted from the data in column 9, Table 8 and Fig. 6 that in every case the concrete which was subjected to freezing and thawing had less strength than the normally-cured concrete 28-days old and considerably less strength than the estimated 60-day strength for normal concrete. Considering all mixes, the concrete which was frozen and thawed had an average strength eighteen per cent less than the normally-cured concrete 28-days old and 26 per cent less than the estimated strength of normally-cured concrete 60-days old.

From the chemical analyses in Table 2 the constitution of each of these cements has been calculated and the percentages of the components are listed in Table 9. In the same table are listed the average absorption data from Table 7, the values of Schurecht's ratio, A/B, from column 10 of Table 8, and the average percentage loss in compressive strength due to freezing. Values for the 1:3:6 mix are excluded in these tabulations. Comparisons show that there was not a great difference in the freezing resistances of the concretes made of these different cements, that made from cement 3M had the lowest absorption, the lowest Schurecht's ratio (A/B), and the lowest loss in strength due to freezing and thawing.

A study of these data indicates that the losses in the freezing and thawing test were greatest for mixes having high water ratios and high absorptions and least for those mixes made with lowest water ratios and having the lowest absorptions. Hence, as Fig. 6 shows, the 1:1½:3 mix of high cement content and low water-cement ratio exhibited the lowest loss, 19 per cent, in freezing and thawing, whereas the lean 1:3:6 mix showed the highest loss, 33 per cent. Making allowances for the discrepancies in the water-cement ratios of the 1:2:4 mixes made with different aggregates, a comparison of the data in Fig. 6 indicates that there

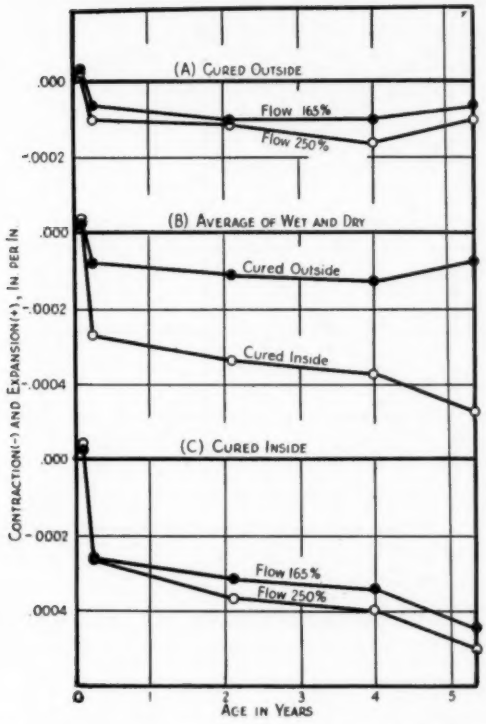


FIG. 7—EXPANSION AND CONTRACTION OF 1:2:4 CONCRETE OF DIFFERENT CONSISTENCIES MADE OF JANESVILLE GRAVEL

was little difference in resistance to freezing of the concretes made with these aggregates.

Results of Expansion and Contraction Tests—The data on effect of outside curing on the linear contraction of 4 x 6 x 24-in. slabs show that at 1 month the mean linear dimensional change in all cases was a small initial expansion (0.00006 to 0.00008) due to curing in the moist closet, but at subsequent periods the outside curing caused a somewhat greater contraction in nearly all cases. The data indicate that the average contraction on concrete made with cement 3M was the least of the average values for the four cements used in these tests. Data from the concrete cured indoors indicated a similar relationship.

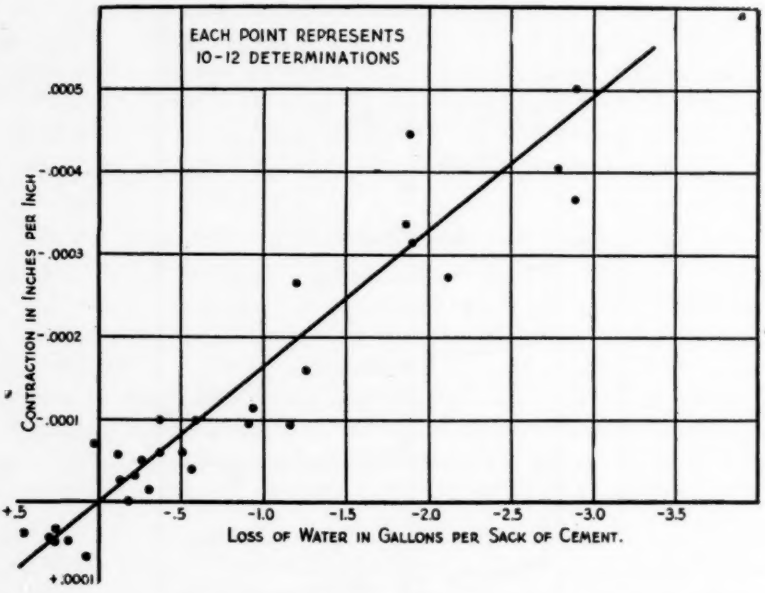


FIG. 8—THE RELATIONSHIP BETWEEN EXPANSION, CONTRACTION, AND LOSS OF WATER IN CURING FOR 1:2:4 CONCRETE

Cement Content of Concrete 5.0 to 5.5 Sacks per cu. yd.

Concrete made of the dolomite showed the most expansion and the least contraction due to outside curing, whereas that made of gravel showed the greatest contraction. In no cases, however, did the average change exceed 0.0001.

The evidence concerning the effect of proportion of cement, is not conclusive, but it indicates that the contraction in curing outside was greatest for the 1:1½:3 mix.

The curves in Fig. 7 (A) and (C) show that the 1:2:4 concrete with a flow of 165 per cent contracted less than the wetter mix with a flow of 250 per cent. Fig. 7 (B) indicates that curing inside caused three to six times as much contraction as the outside curing. At five years contraction of the 1:2:4 concrete due to indoor curing amounted to .04 to .05 per cent.

The most significant relationship obtained in these expansion and contraction tests is that between the percentage of water lost

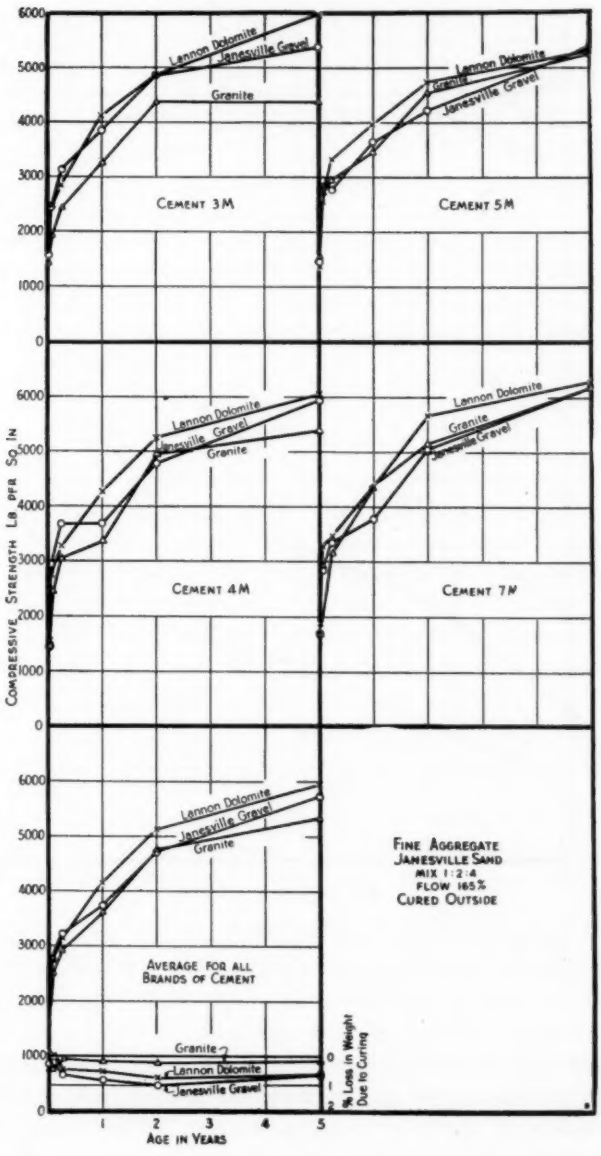


FIG. 9—STRENGTH-AGE RELATIONSHIP FOR CONCRETE MADE OF DIFFERENT BRANDS OF CEMENT AND DIFFERENT AGGREGATES

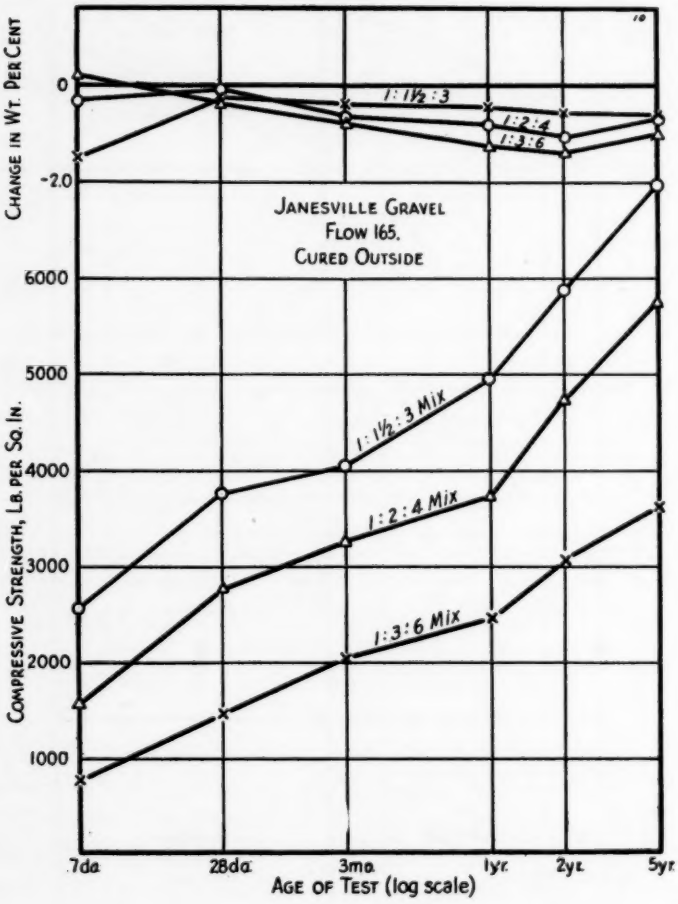


FIG. 10—EFFECT OF MIX ON THE COMPRESSIVE STRENGTH OF CONCRETE AND ON LOSS IN WEIGHT DUE TO CURING

and the change in linear dimension, see Fig. 8. Doubtless most of the discrepancies are due to losses in weight from spalling of the specimens in handling. The line in Fig. 8 checks very closely data obtained at the Portland Cement Association Research Laboratory by M. B. Lagaard.* With this relationship in mind,

*See diagram in Johnson's *Materials of Construction*, 7th Ed., p. 480

one would expect that the contraction would be least for the cement paste exhibiting the lowest absorption. The data herein, though meager, substantiate this view.

Results of Compression Tests of Concrete—The effects of age on the compressive strength of 1:2:4 concrete cured outside are plotted in Fig. 9 to a uniform age scale. The strength-age data for the tests on 1:1½:3, 1:2:4 and 1:3:6 mixes of Janesville sand and gravel are plotted to a logarithmic age scale in Fig. 10. No retrogressions in strength are exhibited in any of these curves, but the retarding influence of seasonal variations in temperature and humidity during the first year are manifested by the corresponding irregularities in the strength-age curves. As in tests of Series A, the data of Fig. 10 show that the strength increased with the logarithm of the age at test.

The curves in Fig. 9 show that the concrete made of Lannon dolomite was slightly stronger than that made of the other aggregates despite the fact that it was mixed with a higher water cement ratio. The concrete from this aggregate also contained a larger proportion of cement than the concrete made with the gravel. The latter fact and the superior bond afforded by the surfaces of the angular dolomite particles undoubtedly account for the superior strength of the dolomite concrete.

The effects of the outside and of the inside curing conditions on the age-strength relationships are shown in Fig. 11. Concrete cured inside exhibited little change in strength after the 3 month tests, and at 5 years had only about 65 per cent of the strength of concrete cured outside.

The data show that 1:2:4 concrete with a flow of 165 percent (w/c = 0.76) had, when cured outside for one year, 35 per cent greater strength than similar concrete with a flow of 250 (w/c = 1.01). At 5 years the mix with 165 per cent flow had 50 per cent greater strength than the wetter mix. Under inside curing the drier mix exhibited from 25 to 35 per cent greater strength than the mix with a flow of 250 per cent.

Since loss in weight during curing is a measure of the desiccating influence of the storage medium on the concrete, it is of interest

TABLE 9—A COMPARISON OF CALCULATED CONSTITUTIONS OF CEMENTS 3M, 4M, 5M, AND 7M WITH RESULTS OF VARIOUS TESTS

Brand of Cement	5M	3M	4M	7M
Per cent $3CaO.SiO_2$	35.1	33.5	21.3	17.6
Per cent $2CaO.SiO_2$	35.0	39.4	47.7	54.2
Per cent $3CaO.Al_2O_3$	14.7	10.1	14.9	10.4
Per cent $4CaO.Al_2O_3.Fe_2O_3$	7.8	7.7	8.2	9.4
Per cent $CaSO_4$	2.6	2.9	3.2	3.5
Per cent MgO	2.3	4.4	2.6	3.6
Ignition loss	2.4	2.3	1.6	1.6
Tensile strength 1:3 Std. Sd. mortar, 28 days, lb/in.^{2}	330	370	386	372
Tensile strength 1:3 Std. Sd. mortar, 5 yr., lb/in.^{2}	233	326	319	246
Tensile strength 1:2 J. Sd. mortar, 28 days, lb/in.^{2}	533	561	575	557
Tensile strength 1:2 J. Sd. mortar, 5 yr., lb/in.^{2}	607	632	664	645
Compressive strength 1:3 Std. Sd. mortar, 28 days, lb/in.^{2}	2750	2490	2720	2450
Compressive strength 1:3 Std. Sd. mortar, 5 yr., lb/in.^{2}	4770	5610	5790	5540
Compressive strength 1:2 J. Sd. mortar, 28 days, lb/in.^{2}	5180	4890	5870	5790
Compressive strength 1:2 J. Sd. mortar, 5 yr., lb/in.^{2}	6950	8050	8520	7690
Gain in neat strength in autoclave, per cent	48	25	24	34
Expansion in autoclave, per cent	.09	.21	.13	.14
Ave. percentage absorption of 1:2:4 concrete	5.35	5.15	5.21	5.17
Ave. Schurecht's ratio	0.88	0.84	0.87	0.87
Ave. loss of crushing strength of concrete due to freezing and thawing, per cent	31	22	24	25
Ave. contraction of 1:2:4 concrete, flow 165 per cent cured outside 4 yr., per cent	0.010	0.0007	0.007	0.008
Ave. contraction of 1:2:4 concrete, flow 165 per cent cured inside 4 yr., per cent	0.036	0.027	0.032	0.040
Ave. crushing strength of 1:2:4 concrete, flow 165 per cent, cured in moist closet 1 mo., lb/in.^{2}	2945	2430	2950	2800
Ave. crushing strength of 1:2:4 concrete, flow 165 per cent, cured outside 5 yr., lb/in.^{2}	5395	5370	5925	6190
Ave. crushing strength of 1:2:4 concrete, flow 250 per cent, cured in moist closet 1 mo., lb/in.^{2}	2125	1445	1780	1790
Ave. crushing strength of 1:2:4 concrete, flow 250 per cent, cured outside 5 yr., lb/in.^{2}	3640	3505	3815	4280

to note the marked discrepancy between the percentage loss in weight of the concrete with flow of 165 cured indoors and the loss in weight of like concrete cured outside. At five years the concrete cured indoors had lost 3 per cent of its weight whereas that cured outside had lost about 0.7 per cent. Losses in weight sustained by the 1:2:4 concrete with a flow of 250 per cent at the

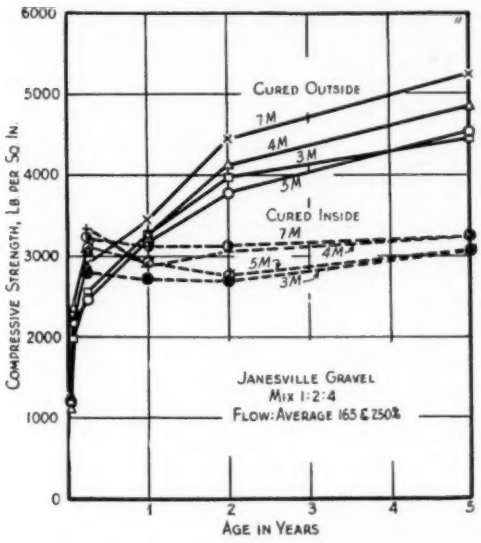


FIG. 11—EFFECT OF CURING CONDITIONS ON STRENGTH OF 1:2:4 CONCRETE WITH DIFFERENT CEMENTS

same age were 3.6 per cent for the concrete cured inside and 1.1 per cent for that cured outside.

The linear relationship between the compressive strength of concrete of 165 per cent flow and the cement-voids ratio is well shown by the curves of Fig. 12. Each point in the diagram represents five tests.

The relationship of compressive strength of concrete to the water-cement ratio, by volume, is indicated in Fig. 13. Each point on these diagrams represents 20 tests.

Influence of Kind of Cement on Properties—Table 9 shows the amounts of the principal constituents in the different cements used in these tests. The percentage of these constituents were calculated from the chemical analysis by Bogue's method.* In Fig. 14 are shown the compressive strength-age curves for the concretes made of each brand of cement. In constructing these

*Paper No. 21, Portland Cement Association Fellowship at the National Bureau of Standards, Washington, D. C.

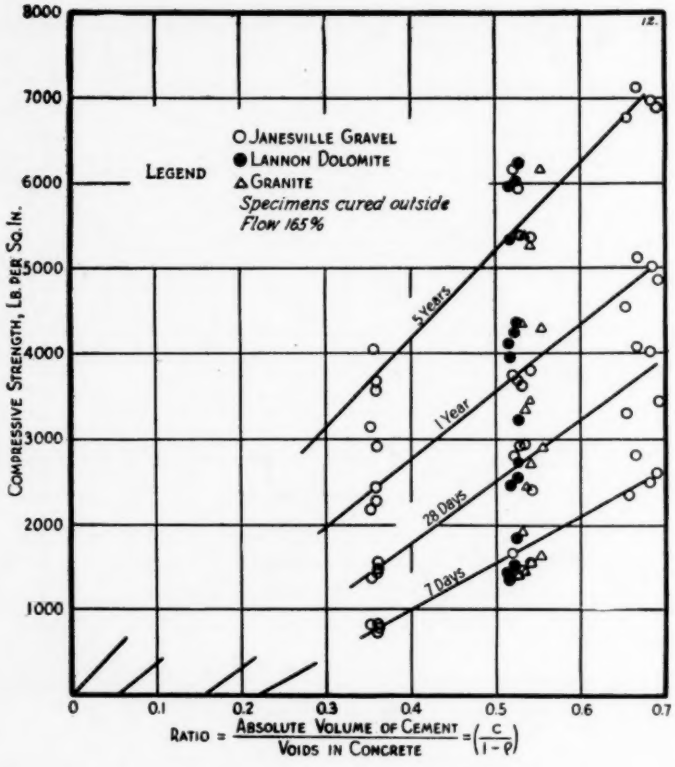


FIG. 12—RELATION BETWEEN THE COMPRESSIVE STRENGTH AND THE RATIO OF THE VOLUME OF CEMENT TO THE VOLUME OF VOIDS IN CONCRETE

curves the strengths of concretes of 165 per cent flow made with the various aggregates and proportions were averaged.

From the data in Table 9 it appears that there were considerable differences in the constitutions of these cements but the discrepancies in the various physical properties are much less. In most cases it will be noted that the cements containing the lower proportions of tri-calcium silicate (3 $CaO.SiO_2$) and the higher proportions of di-calcium silicate (2 $CaO.SiO_2$), brands 7M and 4M, exhibited in these long time tests slightly better strengths

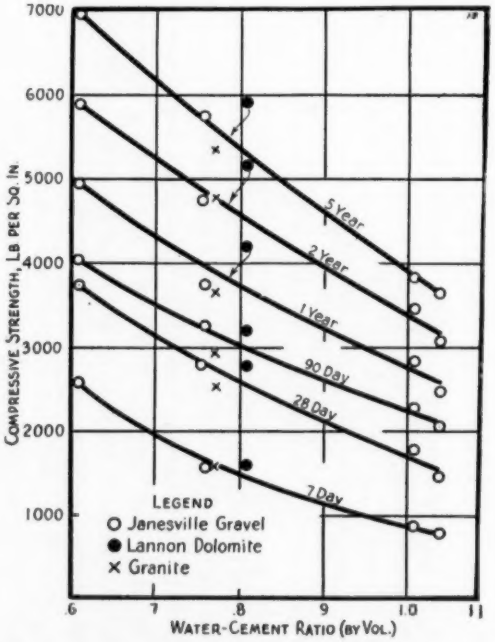


FIG. 13—STRENGTH WATER-CEMENT RATIO RELATIONSHIP FOR CONCRETE OF DIFFERENT AGES

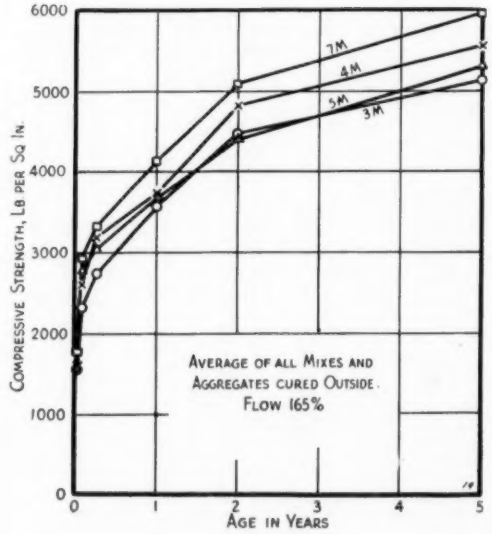


FIG. 14—EFFECT OF BRAND OF CEMENT ON THE STRENGTH-AGE RELATIONSHIP

than the other two brands 5M and 3M which contained higher percentages of tri-calcium silicate.

As previously mentioned, the absorption and the loss in crushing strength due to freezing were slightly greater for the concrete of cement 5M (which was high in tri-calcium silicate and tri-calcium aluminate) than for the concretes made of other cements. The differences in other physical properties listed in Table 9 do not appear to be significant.

Comparison of Compressive Strengths of Concretes of Series A and B with Present Day Concrete—Table 10 has been inserted to furnish comparisons between the average compressive strengths of concrete of Series A (Fig. 3) and Series B (Fig. 13), cured outside, with similar data from the w/c-curves for the moist-cured concretes of Series 210, Fig. 9, p. 158 of the Report of the Director of Research of the Portland Cement Association (1928). The latter tests were begun in 1926. These data indicate that there was not much difference in strengths of concretes of Series A and B but the more recently made concrete of Series 210 was 30 to 50 per cent stronger than the concrete in the Wisconsin tests.

The calculated constitutions in per cent of the cements used in Series A and in Series 210 were:

Cement Used in Series	3 CaO. SiO_2	2 CaO. SiO_2	3 CaO. Al_2O_3	4 CaO. Al_2O_3 FE_2O_3	$CaSO_4$	MgO
A	28.8	40.2	13.8	8.3	3.6	2.5
210	33.0	39.0	12.0	9.0	3.0	2.4

These data indicate that the constitutions of these two cements were quite similar. However, the cement used in Series 210 had a residue on the 200-mesh sieve of 17.5 per cent and was finer ground than the cement used in Series A or Series B. Apparently the difference in fineness of the cements used was a more potent factor in producing the marked difference between the strengths of the earlier-made and the more-recently-made concretes than the difference in constitution. On the other hand, it does not seem possible that the marked discrepancy in the strengths of

TABLE 10—A COMPARISON OF THE AVERAGE COMPRESSIVE STRENGTHS OF CONCRETE IN THESE LONG TIME TESTS WITH PRESENT DAY CONCRETE

Series of Tests	$\frac{w}{c}$, by Vol.	0.94	0.94	0.94	1.36	1.36	1.36
	Age	28 da.	1 yr.	5 yr.	28 da.	1 yr.	5 yr.
Series A Tests* (1910)		1900	3400	4200	900	1900	2400
Series B Tests (1923)		2000	3000	4300			
Series 210 P. C. A. (1926)		3000	5000		1150	2400	

*In order to make the comparison on a basis of strength of 6 by 12-in. cylinders the values from series A, in which 6 by 18-in. cylinders were used, were increased 5 per cent and recorded in the adjoining columns.

these concretes was entirely due to the difference in fineness of the constituent cements.

CONCLUSIONS

1. Portland cement may be stored in a tight container, such as a milk can, for at least 20 years without suffering material change in its strength properties.

2. Tensile strengths of the 1:3 standard sand mortar briquettes cured in water over a 20-year period exhibited retrogression in strength after six months. In the five-year tests of Series B similar retrogression occurred after the 90-day tests.

3. The tensile strengths of 1:3 standard sand mortars cured outside and of those mortars cured in the cellar over the 20-year period increased approximately as the logarithm of the age, but the tensile strengths of the 1:3 standard sand mortars of the later tests cured outside decreased with age after the one-year period.

4. In the five-year tests, mortars of Janesville sand exhibited less retrogression in the tensile strengths of the 1:3 mixes than the similar standard sand mortars. The 1:1 and 1:2 mortars of Janesville sand showed little change in tenacity after curing outside one year.

5. Neat cement mortars exhibited great variations in tensile strength under the curing conditions imposed in both series of tests.

6. In many instances low tensile strengths were undoubtedly due to the presence of surface checks at the minimum cross-sections of the briquettes. In the water-cured specimens of the 20-year tests, leaching appeared to have weakened the tenacity of the 1:3 standard sand mortars.

7. The compressive strengths of neat cement mortars were much more variable than the compressive strengths of the sand mortars. Part of this variability in the neat cement tests of Series A was undoubtedly due to poor bedments and part was due to checking. The compressive strengths of the sand mortars in both series of tests continued to exhibit upward trends with age after the one-year period.

8. The absorptions of the individual specimens of a given concrete, in tests of Series B, were very uniform. There appeared to be little difference in absorption due to aggregate or brand of cement. Absorption appears to increase with the water-cement ratio.

9. All concretes of Series B exhibited losses in compressive strength due to 20 cycles of the freezing and thawing test. Loss in strength due to this treatment was greatest in the mixes having the high water-cement ratios and high absorptions and least in the rich mixes of low water-cement ratios and low absorptions. The aggregates and brands of cement used exercised little effect on resistance to freezing and thawing.

10. Curing the concrete out of doors caused in most cases small linear contractions which for 1:2:4 mixes were usually less than 0.01 per cent at the time measurements were made. The data indicate that either rich mixes or wet mixes contract more than lean or dry mixes. Specimens cured in the dry air of the laboratory contracted three to six times as much as similar specimens cured out of doors and after five years of indoor curing exhibited contractions of 0.04 to 0.05 per cent. Linear dimensional change appears to be directly proportional to the change in water content of concrete during curing.

11. The compressive strengths of the concretes cured in water or cured outside increased approximately with the logarithm of

the age. In the tests begun in 1910 this relationship has persisted through 20 years. In the tests of Series B it has held for 5 years of outside curing despite the evidence from the freezing and thawing tests. The effects of seasonal variations on the strengths of specimens tested during the first year is reflected in the strength-age curves of Series B.

12. The rate of increase in the strength of concrete with age is affected markedly by the humidity of the surrounding atmosphere. Concretes cured in the dry air of the laboratory for five years exhibited water losses of 3 or more per cent and little change in strength after three months. Concretes cured in the cellar where the humidity was less than outside but greater than in the laboratory exhibited a progressive increase in strength but a slower rate of increase than the concretes cured outside.

13. A comparison of the average compressive strengths of the concretes of a given water-cement ratio made in Series A (1910) and in Series B (1923) with later made concrete, Series 210 (1926), indicates that there was not much difference in the strength-producing properties of the cements used in Series A and in Series B; but the more recent tests demonstrate that the early hardening properties of present-day cements have been considerably increased. It is therefore questionable whether the compressive strength of present-day concrete will increase over a period of twenty years in accordance with the logarithmic relationship which obtained in the tests of Series A.

14. Although there was considerable difference in the calculated constitutions of the cements of Series B, the differences in the compressive strengths of the concretes made of these cements were small. Contrary to expectation, in these tests the concrete made of the cement, which was calculated to contain the highest proportion of tri-calcium silicate, was somewhat more absorptive, exhibited greater loss in crushing strength due to freezing and thawing, and slightly less crushing strength in the outdoor tests than the concretes of other cements containing lesser proportions of tri-calcium silicate.

15. Concrete made of crushed Lannon dolomite, though carrying a higher water-cement ratio, was slightly stronger than

that made of other aggregates. This superiority appears to have been due to surface characteristics.

16. With the consistency constant the data from tests of Series B show that the strength varies linearly with the cement-voids ratio.

17. The data from both series of tests confirm the general relationship between the compressive strength and the water-cement ratio. The tests of Series B indicate that this relationship is influenced to some extent by the character of the aggregate.

Readers are referred to the JOURNAL *for June, 1931, for discussion which may develop. Such discussion should reach the Secretary by May 1, 1931.*

Cinders as Concrete Aggregate

Report of Committee 203

BY EINAR CHRISTENSEN*, AUTHOR-CHAIRMAN

Because Mr. Christensen had intimate touch, as engineer and chemist, with the development of an industry dependent on cinders in the manufacture of cinder concrete masonry units, and had given a decade of study to cinder aggregates, their characteristics, properties and the special characteristics and properties which they give to concrete made from them—studies not confined to one factory or condition or locality but to many —he was asked to prepare, as Author-chairman of Committee 203, a report which should summarize available information on cinders as concrete aggregate. His report written, it was submitted to W. D. M. Allan, R. D. Bradbury, Miles N. Clair, Emil Praeger and M. J. Roach—serving as critic members of the committee. Their responses were not perfunctory; were for the most part, detailed and thorough-going—numerous revisions being suggested particularly by Messrs. Allan and Clair. Mr. Christensen acknowledges their assistance in a general strengthening of the report in its revision for publication. At Mr. Christensen's request, the report was also submitted to George A. Strehan. Mr. Strehan's response, too late for consideration prior to the present publication, takes issue in some details with opinions expressed in the report. It will appear later, together with further data and critical comment by critic members of the committee and others. In the report as published Mr. Christensen has had the last word. It does not completely reflect in every detail the views of all the critic-members of the committee, but in general has their approval as an excellent presentation of the subject assigned.—EDITOR.

*Consulting Engineer, New York City.

INTRODUCTION

A COMMON attitude towards cinder (or sand-cinder) concrete is expressed in the anonymous letter in the *News Letter* pages of this JOURNAL for November, 1929—the attitude, briefly, that here is a material to be viewed with suspicion. Its author appears in accord with a British architect (D. B. Butler, 1909) who wrote:

> Judging from the quality of breeze (here "cinders") which may daily be seen delivered to buildings in course of construction, presumably for incorporation in fireproof floors, etc., many a clerk of works must be imbued with a pleasing and childlike faith in the ability of portland cement to stick together any kind of rubbish.

While in the 20 years between these two opinions millions of tons of cinders have been used in making concrete, the skeptical attitude of the anonymous writer to this JOURNAL is somewhat justified. Few data have been published to throw light on the mystery of cinders and their use in concrete.

Text books and technical publications give very little information. Authors generally copy faithfully what has been written before—not altogether condemning cinder concrete, yet warning against its use.

The belief that cinder (sand-cinder) concrete is "dangerous" is not confined to the Anglo-Saxon world. *Die Tonindustrie Zeitung*, for April 25, 1908, reports that the German Department of Public Works recommends the prohibition of cinders as an aggregate for reinforced concrete. The concluding paragraph of the recommendation states:

> Die Verwendung der Schlacke zur Herstellung von sonstigem Beton, sogenannten Stampfbeton, wird nur da zuzulassen sein, wo die Gefahr, die tragende Eisenteile mit derartigem Beton in Berührung kommen oder Menschen von herabfallendem Putzmörtel getroffen werden können, ausgeschlossen ist.

In other words, cinder concrete must not be used where people may be hurt by the falling concrete. This is a sad specification. "Menschen von herabfallendem Putzmörtel getroffen werden können." It should be very unsafe to live in New York.

As cinder concrete is now being used in the United States and in Canada, in masonry units, equivalent to approximately 500

million brick per year, and sand-cinder concrete in floors and fireproofing of many noteworthy buildings, it is timely to attempt evaluation of the knowledge we have of these materials. It is discomforting that one large city permits what another flatly condemns—and that one engineer places thousands of cubic yards of a material which another believes doomed to failure.

The common attitude towards cinder concrete, particularly sand-cinder concrete made on the job, is not surprising. The contractor is not inspired by the looks of the aggregate, nor by a "1:2:5" specification. He pays less attention to this than to stone concrete; the aggregate is ordered over the telephone and passes directly from the truck to the mixer without much inspection. Twenty thousand cubic yards may be placed in one building in this manner. The result, sometimes good and sometimes bad, is generally not known in advance. Aside from extraneous matter the cinders look rather dirty and unreliable, because cinders are always wet, their color and the true nature of their particles being disguised.

To see what cinders look like, a sample should be dried and split by a standard sieve analysis. It will then be found, generally, that the aggregate contains a range of sizes and consists of particles which have sufficient strength to offer concrete-making possibilities. By the same simple process it is possible to observe the peculiar shape and cellularity of the particles. How many engineers make sieve analyses of cinders? Even while using larger quantities of them in the floors of one building.

Here the cinder unit industry has contributed to our knowledge. Factory-made cinder units could be studied readily with reference to their general characteristics—and that naturally suggested a study of the raw material and the quality factors. In preparing this report, the writer is relying on ten years' experience in the manufacture of cinder concrete building units. In the light of this experience he attempts to interpret the work of other investigators of cinders and the resulting concrete. It is hoped facts will be established tending towards the making of better cinder and sand-cinder concrete, and a better understanding of the material.

The committee has been directed to present in summary a survey of the whole field, from the nature of cinders to the characteristics of the resulting concrete. Within the last few years, certain problems of cinders and cinder concrete have been subjected to careful study; prior to that, the matter has been treated so broadly and repetitiously that a thousand references may well be condensed to ten or less.

While this committee's assignment is in reference to cinder aggregate, the report is concerned chiefly with the type of concrete in which cinders are used as the sole aggregate—the type of cinder concrete at present used most extensively in precast masonry units. This must be clearly understood, inasmuch as the term "cinder concrete" is frequently applied to "sand-cinder concrete" (1:2:5 or so) as used in floor arches and the like. The definitions used in this paper should be carefully noted. When the paper deals with cinder concrete it is mainly because the writer's experience has been with the cinder unit industry where cinders alone are used as aggregate. The cinders, however, are the important and typical constituent of both types of concrete—the nature of this raw material and the character which it gives to the concrete are of primary interest.

DEFINITIONS

The following definitions may not meet the wholehearted approval of all readers, but they will apply throughout this paper.

Cinder Concrete: A concrete made from an aggregate composed of cinders (commonly used in the manufacture of cinder concrete masonry units—the aggregate usually prepared by crushing).

Sand-Cinder Concrete: A concrete made from a mixed aggregate of sand and cinder (commonly used under the name "cinder concrete" in floors, fireproofing and the like—the sand generally considered the fine aggregate; the cinders sometimes screened but rarely crushed).

Cinders: The residue, containing more or less carbonaceous matter, from high temperature combustion of coal or coke— known as "industrial cinders," "boiler cinders" or "steam cinders," to the exclusion of the residue from domestic furnaces. Forced draft is generally applied in the combustion process.

Ashes: The residue, containing more or less carbonaceous matter, which remains after the incomplete combustion of coal, coke, wood, or the like. This is an inclusive term, most frequently applied to the residue from domestic furnaces.

Ash: The inorganic residue which remains after complete combustion of coal, coke, wood or other carbonaceous matter. This may be considered a "laboratory term."

Clinker: That part of the cinders which has been partly or completely fused during the combustion process. The clinker may be dense or more or less cellular.

The present report is primarily concerned with cinder concrete as here defined, and is concerned *only* with cinders as an aggregate. Cinders, as here defined, contain clinker (fused), ash (unfused), and unconsumed carbon (coke or coal). This raw material is obtained from industrial plants, gas works, locomotives, and similar sources. It is not to be confused with the residue from small domestic furnaces, which here comes within the category "ashes," and which is not used in making good concrete.

Confusion arises when the term "ashes" is also applied to the finer (partly unfused) particles of cinders. The physical character of these fines in the residue from high temperature combustion, is different from the soft and dusty ashes of the domestic furnace, and is generally of real value in making concrete. Perhaps some of the skepticism regarding cinder concrete resides in the fact that even learned men associate the material with the unpleasant refuse of their basement fire.

It is of primary importance to the maker of cinder concrete (or of sand-cinder concrete) to know that his raw material is coming from a source where a large amount of coal is burned. This usually entails uniformity of coal and uniformity and efficiency of firing, with the result that the cinders do not vary greatly from day to day. In most places, cinders have ceased to be a waste product; they are now considered a commodity, and a large producer takes care to prevent the use of his cinder pile as a general dump.

No general differentiation is here made between anthracite and bituminous cinders. While they differ in their physical character, they are both suitable for concrete when meeting the requirements here proposed for cinders. The "sulphur controversy" has given rise to the idea that anthracite cinders are superior, and certain specifications exclude bituminous cinders.

Evidence at hand does not justify this attitude. As a matter of fact, the writer prefers a well prepared bituminous cinder because the particles are usually stronger and because the major part of the combustible matter is usually present as coke.

In England, "breeze" and "coke breeze" complicate the terminology. The former is broader in scope than the term "cinders;" the latter applies to the small coke produced by coke ovens and gas works. The term "furnace clinkers" comes rather close to the definition of "cinders" proposed here. In Germany the word "Schlacke" applies to both cinders and blast furnace slag. When designating the former it should be "Kesselschlacke" —in case of the latter "Hochofenschlacke." The writer has encountered several misunderstandings (notably in patent cases) because of erroneous translation of these terms.

CINDERS

The inorganic residue from the combustion of coal is composed largely of silica, alumina, lime and iron, with smaller quantities of magnesium, titanium and alkali compounds. A small amount of sulphur compounds is generally present, mainly in the form of sulphates and sulphides. The combustion process is practically always incomplete, and the residue therefore contains carbon as coal or coke or both.

A complete chemical analysis of cinders is of very little value in judging the concrete-making possibilities of the raw material. The maker of cinder concrete is more interested in the physical character, and this may readily be judged by an experienced observer. An analysis, if at all desired, may be confined to a determination of coal, coke, sulphates, sulphides, and (perhaps) lime.

Cinders from well fired bituminous coal contain their carbon mainly in coke, formed by distillation on the grate, while anthracite cinders may contain considerable unchanged coal. Average percentages are of little interest, but the average combustible content of industrial cinders of the United States may well be close to 30 per cent. The frequently-encountered specification for cinder concrete, that the cinders "must be free from unburned coal," would eliminate most of the raw materials which have come

to the writer's attention. While such a specification is not justified, the carbon content of the cinders deserves, nevertheless, some attention.

Effect on Strength: When a specimen of cinder concrete is broken in compression, it will usually be observed that particles of coal are exposed as such, while particles of coke are fractured. This indicates that the smooth surface of the coal does not adhere to the cement, and that the coke has less strength than other particles of the aggregate. No definite relationship has been established between carbon content and strength—the main reason being that it is difficult, in comparative tests, to control the variable of consistency when varying the amounts of dense coal or porous coke.

Effect on Fire Resistance: It is natural to expect that the presence of coal and coke in cinders will reduce the fire resistance of concrete. Such an effect is, however, surprisingly small. Free combustion cannot take place when a piece of coal or coke is well protected in properly made cinder concrete. The only apparent effect is a distillation of bituminous coal when a certain temperature is reached—but that is a factor of minor importance in view of the high insulating quality of well-made cinder concrete. The effect is different when the concrete is so loosely packed, or the aggregate so void of fines, that fire may gain free access to the interior.

In factory-made cinder concrete units, the effect of carbon content upon fire resistance has been studied in a series of tests. The units were 8 x 8 x 16-in. hollow blocks, 2 cores, about 27 per cent air space, 2-in webs, having a compressive strength at 28 days ranging from an average of 825 lbs. per sq. in. of gross area for one batch to an average of 1156 lbs. for another. In preliminary tests, small panels (32½ in. square) were built of these units. The combustible content of the aggregate consisted, approximately, of ¼ anthracite coal, ¼ bituminous coal, and ½ coke—the extremes being an aggregate with about 20 per cent and another with about 75 per cent combustible content. The latter was prepared from nothing but coal and coke, the percentage being the result of chemical analysis, deducting ash. Considering these two extremes, the two panels had approxi-

mately the same endpoint (250 deg. F. above initial temperature of unexposed face) while the compressive strength after fire test had decreased 31.7 per cent with the lower carbon content, and 55 per cent with the higher carbon content. The panels behaved remarkably well, and even the "pure coal-and-coke block" did not "catch fire." The bituminous coal of this block suffered from distillation, evidenced by gas and smoke. At the same time, this distillation apparently retarded the heat flow through the wall. Later, test was made on a large panel (about 10 ft. x 11 ft.) which was subjected to a load of 80 lbs. per square inch during test. Three types of cinder blocks were included, made as before, and containing three percentages of combustible matter,—20, 35 and 45 (by weight of aggregate) respectively. After three hours fire exposure the endpoint was not reached. Next day the wall was loaded to failure, and failed in the 45 per cent carbon block at approximately triple load, or about 250 lbs. per square inch. On the basis of these data, the Underwriters Laboratories incorporated in their specifications for hollow concrete building units: "When cinders are used the average combustible content of the mixed fine and coarse aggregate shall not exceed 35 per cent by weight of the dried mixed aggregates."

Effect on Permanence: This has been given particular attention in England where instances have been reported of expansional movements of cinder concrete attributable to the presence of certain types of coal in the cinders. The writer has seen very few such instances in the United States, and only in one case (that of a poorly burned bituminous cinder) could it be ascribed to the presence of coal. Bearing in mind the foregoing definition of "cinders," it is believed that the phenomenon is of less interest here than in England where the material is less clearly defined.

The problem has been studied by D. B. Butler[5] and F. M. Lea.[15] The investigation by Lea is the most recent and the most comprehensive; it is part of the interesting work on cinders by the British Department of Scientific and Industrial Research.

According to Lea,[15] coals which are dangerous in concretes are so because of expansional movements which they produce during the setting period and early life of the concrete. The effect depends upon the nature of the coal; the amount present does

not by itself afford a test of the soundness of the material. The amount of moisture or of oxygen taken up by the coal are the characteristics to consider. With increasing rate of absorption the coal becomes dangerous.

Lea develops a test to determine the nature of the coal in cinders in its relation to the permanence of the concrete. It consists in making a 3-in. pat of 1 part binder (equal parts by weight of cement and plaster of paris) and 3 parts of finely ground (No. 76) cinders, gauged with water to a plastic condition. The pat is kept in a moist atmosphere for 3 to 4 hours, thereafter left totally immersed in water, and is examined after one to four days. If fine radial cracks appear, running from edge to center, or if the edges lift, or if the pat becomes loose, the cinders are unsound.

Cinders from high temperature combustion should contain very little coal of this nature, except due to accidents in firing the boiler. Such cinders, when bituminous, should contain the major part of their carbon in the form of coke. Anthracite coal appears to be harmless. Lea states: "If the coal has been treated to a sufficiently high temperature, its dangerous properties will be largely or entirely removed."

Sulphur Compounds

Various sulphur compounds may be present in cinders. In the unburned coal (if any) sulphur may remain as pyrites, marcasite or organic sulphur compounds; in the ash or clinker the sulphur may be present as sulphates or sulphides. Most cinders which have come to the writer's attention over a period of ten years contain less than 1 per cent of sulphur (S) in these compounds. They are present in the coarse as well as in the fine particles of the cinders.

When considering the possible effect of such combinations of sulphur, it should be borne in mind that they are not present in readily reactive form, but usually embedded in the coal or clinker or as an occasional large piece of pyrite. The behavior under such conditions is quite different from that of pure powders, and "laboratory deductions" do not always apply.

Corrosive Effect: It is well established that corrosion has occurred in several instances of reinforcing steel in cinder concrete

or of pipes and the like in cinder fill. Such instances, however, are many times outweighed in number by the performance of sand-cinder concrete or of cinder concrete building units where the behavior towards steel could be observed.

Corrosion of steel in cinder concrete has usually been explained as due to "sulphur," the supposition being that this element (seldom more clearly defined) by oxidation would form sulphuric acid which would then attack the steel. The chemical terms so appealed to many concrete engineers that this corrosion process became an axiom of the art. Instead of "sulphur" the word "acid" is frequently used to indicate that something is bound to happen when cinders are used—not merely corrosion, but staining as well.

It may be well, for a moment, to consider stone concrete. It is not uncommon to find corrosion of steel embedded in this material, but such instances are always explained by the fact that the concrete is poorly made or that the steel is improperly protected—a matter of mixing, making and placing—and no one would consider it necessary to assume chemically active elements in the aggregate. It would appear reasonable that the same explanation is at least to some extent applicable to concretes made with cinders as an aggregate.

Prof. C. L. Norton[1] came very close to this explanation in his well-known investigation of sand-cinder concrete. After pointing out that "sulphur" cannot be the cause in the "presence of the strongly alkaline cement," he concludes:

> There is one cure and one only: *Mix wet and mix well.* With this precaution I would trust cinder concrete quite as quickly as stone concrete in the matter of corrosion.

While Professor Norton absolves the "sulphur" of all blame and reaches the general conclusion that the protection of steel is a matter of the quality of concrete, he does attach some significance to the iron oxide, or rust, which may be found in cinders. He states that "a rust spot on reinforcement was invariably coincident with a void in the concrete or a badly rusted cinder." and concludes that this iron oxide may cause trouble. The explanation seems rather strange. It is far more plausible that the voids (permeability) were the sole cause, as they would allow moisture

to attack the reinforcement as well as the iron present in the cinders. The rust spots would naturally be coincident.

That the protection of steel in cinder concrete is primarily a matter of quality of concrete (mixing and placing) rather than a matter of some constituent of cinders, is supported by the evidence of several observers. Reference is here made to the investigation of sand-cinder concrete reported by Perrine and Strehan,[10] together with the discussion following that paper, notably by A. L. A. Himmelwright. Perrine and Strehan conclude as follows:

> Anthracite cinder concrete (1:2:5), well mixed, cast in a viscous to wet consistency, constantly stirred and mixed during placement, in such a manner as to coat the reinforcement thoroughly with mortar, will not cause the corrosion of embedded steel.

Millions of cubic yards of sand-cinder concrete have been placed in New York city. It is therefore significant when Rudolph P. Miller[7] (former Superintendent of Buildings, New York City) states (1911) that he has yet to hear of the first instance of corrosion of steel in this type of floors. While the cinders of New York are, most commonly, from anthracite coal, they nevertheless contain sulphur compounds to practically the same degree, and of the same nature, as cinders in other parts of the country. If corrosion is a matter of chemically active sulphur compounds, there would presumably be a large number of instances on record in New York. On the other hand, if it is a question of quality of workmanship, it is to be expected that the New York contractors would gain more than usual experience in the making of sand-cinder concrete, with the result that a better grade of concrete would be produced.

Factory-made cinder units represent well made cinder concrete —the cinders being crushed, well mixed with cement and water, and molded under pressure. These units are made throughout the United States and Canada—involving all kinds of cinders, anthracite, bituminous, Pennsylvania, West Virginia, etc., etc. The aggregate is all cinders, with no sand added. Therefore, if the corrosion of steel in cinder concrete were due to chemically active elements of the cinders, it would certainly have occurred somewhere and at some time in case of these cinder units. More so than in case of concrete where part of the aggregate is sand.

The writer has examined a great number of nails which for years had been embedded in this kind of cinder concrete—and also some cinder concrete lintels containing reinforcing rods. In no instance has he found sign of corrosion in the concrete. When a nail is driven into cinder concrete some of the aggregate particles are crushed and the dust is forced into adjoining pores, filling them and giving a firm hold on the nail. Fresh surfaces of cement are laid open, and if moisture gets into the concrete, further hydration takes place, with the result that the wound heals and the nail becomes cemented together with the surroundings.

Based on the above observations it appears unnecessary to assume any chemical reaction of sulphur compounds in order to explain occasional corrosion of steel in cinder concrete. It is possible that sulphates or other soluble salts of the cinders may accelerate rusting if moisture and air get into contact with the steel of the concrete. This is indicated by a recent investigation by F. L. Brady.[16] The method employed by Brady may, however, be subject to some criticism. The aggregates used included "coal clinker," "coke breeze," "furnace ashes" and "clinker from refuse destructor." These were ground to pass No. 20 mesh and made into mortar blocks (1:6). After one week, the mortar blocks were crushed to pass No. 10 mesh, and this dried mixture of aggregate and cement was used to make three series of tests. Steel strips were surrounded by the crushed mortars and various exposure to moisture and humid air were tried. As a basis for comparison of corroding power a comparable mixture was prepared using standard sand as aggregate. These tests showed (practically) an absence of corrosion in case of the sand mortar, and increasing corrosion in case of increasing amounts of sulphur compounds in the other mortars. This indicates that the corrosion process, under the conditions of the test, is accelerated by the soluble salts present as well as by the moisture adsorption, the latter being particularly large in cinder mortars.

In only one test series did Brady embed his steel in "cinder concrete"—and this concrete appears to have been of a somewhat unusual design. It is reported to consist of 1 cement, 2 "clinker," finer than No. 10 mesh, 4 "clinker" coarser than ¼-in. and finer than ¾-in., all parts by weight. It will be noted that inter-

mediate sizes are absent, indicating a rather inferior concrete. Furthermore, the steel was protected by only 1 in. of concrete. Corrosion occurred to varying degrees when the steel was examined after about two months, increasing with increasing content of sulphur compounds.

It should be observed that this effect of sulphur compounds is quite different from the effect commonly ascribed to "sulphur." It is a matter of accelerating the corrosion process when such process is possible because of improper protection of the steel in the cinder concrete (or in cinder fill). It is not a matter of an "acid attack" which would take place regardless of the quality of concrete. Such oxidation of sulphides to sulphuric acid is highly improbable, and even if acid were formed, certainly it would not attack the steel inasmuch as it would be neutralized immediately by the bases present in concrete.

The observations of the performance of reinforced cinder and sand-cinder concrete in the field are not upset by the above quoted laboratory test. The fact remains that millions of cubic yards of such concrete have given excellent protection to the embedded steel. The quality of the concrete (mixing and placing) is the determining factor. If the steel is not protected, corrosion is probably accelerated by the presence of soluble salts. Pipes and the like in cinder fill generally have no protection against air and moisture.

The steel must be protected. In floors, the sand-cinder concrete must have a waterproof top covering. The concrete should be well agitated and mixed while placing, and it is well to coat the steel with cement mortar. In masonry walls, cinder concrete building units must be protected against water forced in from the outside and the usual care must be taken in construction details at spandrel beams and the like where, otherwise, leakage may occur.

Expansional Effect: Dr. H. Nitzsche[13] has investigated the effect of sulphates in cinders on the permanence of cinder concrete. His work is concerned with the possible reaction between these sulphates and the lime of the cement. The reaction, if any, results in a volume change. Nitzche used a cinder containing "insoluble" $CaSo_4$ and soluble $MgSo_4$, K_2So_4 and Na_2SO_4. Start-

ing with a raw material rather high in these sulphates (containing .93 per cent "insoluble" and .81 per cent soluble) he reduced the percentages by repeated washing or increased them by adding the salts in powder form. The significance of tests involving the latter may be questioned, inasmuch as sulphates are not present in cinders in such readily reactive form. With an aggregate finer than ¼ in., and with 66.7 per cent retained on No. 16 mesh, he produced small Bauschinger prisms of 1 cement: 2 sand: 7 cinders. These prisms were measured for volume change under water, in air, as well as in sealed glass containers where the original moisture was maintained. On the basis of this investigation, Nitzsche reached the conclusion that for permanent concrete, the cinders must contain not more than 1.0 per cent $CaSO_4$ and 0.2 per cent soluble sulphates. He found the volume change to be considerably less when blast furnace cement (Hochofenzement) was used instead of portland cement.

Lea also investigates[15] the expansion of cinder concrete which may be ascribed to the sulphates. He goes further than Nitzsche in that he produces higher sulphate contents by fusing or sintering the admixed sulphates with the cinders. In this form, the effect of the sulphates is far less marked than as fine powders. Lea concludes that the limits proposed by Nitzsche are unnecessarily severe. He says of soluble sulphates (page 65) that "it is concluded that the average content of alkali, and magnesium, sulphates present in breezes and clinkers is small and may be neglected as of little importance." Of the effect of calcium sulphate he states (page 66) that "on the basis of these results it can be concluded that a limit of 0.4 per cent sulphate as sulphur trioxide (= 1.00 per cent SO_3 = 1.70 per cent $CaSo_4$) can be safely tolerated, and that in many cases much higher percentages are unlikely to cause trouble."

The ash and clinker may contain a small amount of sulphide and the unconsumed coal may contain Pyrite (FeS_2). The latter, however, decomposes at a temperature below 1290 deg. F., which is below the temperature of the furnaces considered here; it can therefore only be present due to accident in firing and the like. Lea produced oxidation and/or hydrolysis of calcium sulphide in cinder concrete when the sulphide had been added as

a fine powder—and he suggests a limit, in this case, of 0.4 per cent sulphide sulphur. However, when the sulphide was fused with a clinker it was found to be efficiently protected against hydrolysis by high pressure steam and apparently also protected against air oxidation. No effect was found with ferrous sulphide. Lee proposes a limit for "all sulphur other than sulphate sulphur" of .75 per cent S.

While the writer has observed occasional expansional movements of cinder concrete during its early life, he has never been able to identify definitely the cause of the phenomenon. The most common cause appears to be that of slow hydration of free lime (discussed later). Having no contradictory evidence on record, it is well to accept the limits proposed by Lea. Cinders, as previously defined will nearly always come well within these limits.

Lime

Small lumps of free lime may occur occasionally in cinders. Usually the lime is fused with the silica and alumina of the ash to form totally inert compounds. Small lumps of free lime may remain, partly hydrated or otherwise combined on the outside so that the result is a nucelus of CaO surrounded by a rather dense shell of hydroxide, carbonate, silicate or the like. Moisture may eventually penetrate this shell and hydrate the CaO. In a cinder concrete, the result is like that of "lime spots" in clay brick. The phenomenon is known as "popping." It is not a frequent occurrence and is evidently confined to certain types of coal. In cinder unit plants using steam curing, hydration lime spots will generally take place during the steaming process.

Contaminations

According to the definition, we are considering as a raw material only those cinders, which are the residue from high temperature combustion of coal and coke. If wood ash, pulp or the like are burned with the coal or coke on the grate, the cinders are contaminated. In selecting cinders for concrete, it is necessary to know something about how they are made. The specification should be written accordingly.

Cinders may be contaminated after they leave the boiler. Fortunately, most large producers of cinders have realized their

commercial value and the cinder pile is no longer considered a general dump. It is essential for the maker of cinder concrete to be sure that the cinder deposit is kept clean. The writer knows of a few scattered instances of caustic soda, molasses, and the like, being dumped with the cinders. These contaminations were readily discovered because the concrete would not harden. Particular attention is required where cinders are obtained from chemical plants.

Stray iron may be considered a contamination. It may occur as scalings from the fire place, as nails, bolts, grate bars, etc. In cinder unit plants this iron may be removed, with a fair degree of efficiency, by a magnetic separator. It does no general harm except when it gets into the equipment.

General Physical Character

The structural strength of the particles of cinders depends upon the nature of the coal and the temperature of burning. The clinker is the strongest part; it is formed by fusion of the ash constituents of the coal when subjected to the high temperature of the furnace. The fusing point of coal ash increases with the amount of alumina and the sum total of silica and alumina, and decreases with the sum total of iron oxide, lime and magnesia. A fusible ash will produce a dark clinker containing a relatively small amount of friable or fine material. A less fusible ash may yield laminated or honeycombed material which possesses less structural strength.

Because of the high fusibility temperature of anthracite, cinders from this coal do not contain clinker as thoroughly fused as cinders from bituminous coal similarly burned. The clinkers from anthracite are structurally weaker. The permissible maximum size of anthracite cinder aggregate is therefore usually less than for the average bituminous cinder.

Anthracite cinders usually contain more coal than bituminous cinders. While this coal is less objectionable than bituminous coal in concrete (it is chemically more stable and physically stronger) it nevertheless does not improve the concrete because of its smooth surfaces. As far as its effect upon the cellular structure of the concrete is concerned, the density of this anthracite coal is offset by the relatively high porosity of the clinker.

On the other hand, in bituminous cinders the carbon (present mainly as coke) increases the cellularity while the clinkers are relatively dense.

The weight depends upon the nature of the cinders (coke, clinker, etc.) and the grading from coarse to fine. Averages are of little interest. In one series of tests, a typical Eastern anthracite cinder (crushed to maximum ¾-in., F. M. 4.1) weighed 55 lbs. damp loose and 61 lbs. dry rodded per cu. ft. In another series, a bituminous railroad cinder weighed 53 lbs. damp loose and 60 lbs. per cu. ft. dry rodded. In a third series, a bituminous cinder from an electric power plant (crushed to maximum ¾-in., F. M. 4.5) weighed 64 lbs. damp loose and 71 lbs. per cu. ft. dry rodded. This cinder had about 20 per cent combustible content. With 60 per cent coke(weighing 42 lbs. damp loose) the cinder weighed 54 lbs. per cu. ft. dry rodded, showing the effect of the coke. With cinders from one source, the weight has been found to be of some value as a check on the uniformity of the product.

A SPECIFICATION FOR CINDERS

In proposing this specification for cinders, the writer has in mind both the raw material used in sand-cinder concrete (usually without crushing or other preparation) and that used in the manufacture of cinder building units (where it is further prepared by crushing, etc.) or in other types of cinder concrete.

Cinders, when used for concrete, shall be the product of high temperature combustion of coal and/or coke—known as "Industrial Cinders," "Boiler Cinders," or "Steam Cinders," to the exclusion of the residue from domestic furnaces. The cinders shall be well burned, free from foreign matter, and so graded from coarse to fine as to produce a cinder concrete (or sand-cinder concrete) meeting the strength requirement of the building code. The cinders shall contain not more than 35 per cent of combustible content by weight, nor more than 0.45 per cent sulphur (*S*) as sulphide, nor more than 1.00 per cent sulphur trioxide (SO_3) as sulphate.

The experience of the writer has shown that if the cinders meet the first part of the specification, they will also meet the requirements for unconsumed carbon (here adopted from the specifications of the Underwriters' Laboratories) and those for sulphur compounds (here adopted from the recommendations of Lea). It would then appear unnecessary to impose these further

tests on the producer. Accurate sampling of cinders for chemical analysis is extremely difficult—it is more conducive to the making of good concrete to expend time and effort in obtaining cinders from a reliable source, free from contaminations.

As fas as a general specification is concerned, a strength requirement is usually sufficient to govern the quality of cinder concrete, provided the raw material is "cinders" as here defined. It is not necessary to go into details of the physical makeup of the aggregate, or the design of mix, if the strength is produced. The specification as to maximum size naturally depends upon the purpose for which the concrete is used and should not be included in a general specification. The peculiar characteristics of cinder concrete, which depend upon its cellular structure, cannot be anticipated in a general specification for the raw material; they are developed to a varying degree dependent upon the admixture of other aggregate (sand) and the methods of making the concrete.

The writer knows of few suitable specifications developed by other investigators. Selecting a text book at random, the following specification is found:

> The standard specification for cinders states that they shall be of hard, clean, vitreous clinkers, free from sulphides, unburned coal, or ashes.

No such cinders are ever found. The author of this specification might as well have said: "Cinders are never to be used." The fact remains that cinders have been used with excellent results, although violating this and other specifications. The writer is of the opinion that a specification governing this or other materials, should be liberal enough to be practical. Otherwise it is not enforced.

JUDGING CINDERS FOR CONCRETE

Notwithstanding all exactness of the science of concrete making, judgment is still an important factor. When using cinders as an aggregate, it is well to be able to judge the quality of the raw material without having to resort to elaborate physical and chemical tests.

The manufacturer of cinder concrete building units usually contracts for his cinders, getting them from the same source

over a long period of time. He is more assured of uniformity than the user of an occasional few thousand yards of cinders for sand-cinder concrete in floors or fireproofing. The latter has to depend, largely, upon a cinder broker, and fluctuations in the cinder market make it difficult sometimes to be very exacting in the demands. The contractor may have to "take what he can get;" then the matter passes beyond the realm of exactness into the land of chance which cannot possibly be charted here. It is appreciated that sand-cinder concrete has occupied a peculiar position because of this element of uncertainty. No one would accept sand and gravel from "any pit," and if there is no choice among cinders, the writer is the first to advocate the use of something else.

It is certainly not good engineering practice to specify an aggregate by name only and then accept anything which may be pressed within the category. The foregoing specification is not as indefinite as that; the aggregate may, perhaps, not be judged to its second decimal—nevertheless it is fairly well defined.

An idea of the quality of a cinder may be based upon the following knowledge and observation:

1. Ascertain whether the producer has a steady and large output of cinders —employing forced draft or other means of efficient combustion. Much knowledge may be gained from the daily records of the producer, frequently furnishing chemical analysis.

2. Examine the pile of cinders to see if it contains any rubbish. If so, the pile may have been used as a general dump, and the cinders are not reliable.

3. From cinders from a chemical plant, obtain an average sample of about 1 cu. ft. Divide this by quartering, and boil a fraction with distilled water. Test the water extract with litmus paper. The paper should turn slightly blue. If it turns strongly blue—or red—the cinders should be analyzed to determine the cause.

4. Obtain an average sample as above. This sampling is done by taking shovel-fulls from edges, middle and top of pile. Mix well, quarter, re-quarter, and examine as follows:

4a. Dump these cinders in a bucket of water. The fine ash and the lighter coke will float on the surface, and the amount and the nature of these may be judged. A fair amount of each is desirable—the coke contributing to lightness and cellularity, the ash forming mortar with the cement.

4b. Drain the water from the bucket and compare the appearance of the washed cinders with that of the unwashed. The washing makes it possible to

determine with a fair degree of accuracy the relative amounts of coke, coal, fines, porous, flaky, or fully vitrified clinker. The grading may be observed.

4c. The structural soundness may be judged by breaking down the larger particles.

5. Determine the weight per cubic foot, damp loose, drained of free water. It should be not less than 35 lbs., preferably about 45 lbs. Pass it over a screen with ¼-in. mesh thoroughly shaking the screen. The amount retained on the screen should be at least 50 per cent by weight.

These suggestions may appear very crude to the engineer who is accustomed to deal with other aggregates where (theoretically at least) far more exact rules may be laid down. Surely, it is a far cry from judging cinders in a bucket full of water to obtaining a stone aggregate with a fully defined pedigree. Nevertheless, the use of the bucket has been very useful in revealing much regarding the nature of the cinders at hand. It is a matter of experience.

TESTING CINDERS FOR CONCRETE

In the foregoing the writer has suggested a shortcut for judging the quality of cinders for concrete. When it is a matter of examining the raw material for a cinder unit plant (which may well use 25,000 yds. or more per year), or for a large job of cinder or sand-cinder concrete (which may well require that amount in the floors alone), a definite effort should be made to obtain the cinders from one reliable source. They should then be subjected to more exhaustive tests.

If possible, cinders should be obtained from the source in question over a period of at least one week before signing contract. Daily test specimens should then be prepared in accordance with standard practice so that the uniformity and the quality of the resulting concrete may be determined. A week's average from a reliable source may well be taken as representative of the output.

It should not be necessary to emphasize that a trial run is of little value unless all factors are observed. The grading of the aggregate must be checked, the water content of the cinders must be known (it varies and is frequently very large), the amount of added mixing water must be determined, the method of mixing and making must be uniform. This is obvious for stone concrete; for some reason the rules are not always observed with cinder concrete.

The cinders should be submitted to a recognized laboratory for the following tests:

1. Determination of total combustible content. Nature of coal to be reported. Information may be gained from producer's records.

2. Examination of coal in accordance with method proposed by Lea and previously described. This test may be made upon the cinders or upon the coal picked by hand; the latter method will naturally emphasize results. The test is not necessary with anthracite.

3. Determination of total as well as water-soluble sulphates, quantitatively and qualitatively. Determination of sulphides.

4. Examination for impurities, visual as well as in water extract. Reaction of water extract to be reported.

5. Standard sieve analysis on representative samples to determine daily averages.

The results of this examination may then be judged in the light of previous discussion.

MAKING CINDER CONCRETE BUILDING UNITS

This chapter is concerned with concrete made with an aggregate of cinders only. It is primarily concerned with making this concrete in masonry units—at present the most important field for "self-contained" cinder aggregate.

The cinder concrete here considered is manufactured on pressure or tamping machines. It is of such consistency that the freshly molded unit retains its shape when carried from the machine, resting on a "pallet." The problem is therefore quite different from that of poured concrete, to be discussed later. The relatively dry consistency of machine-made cinder concrete units is made necessary by the manufacturing method. A type of concrete has been evolved which is not to be considered merely a compromise, but as having distinct advantages, among others a most interesting surface texture.

The standard unit is the 8 x 8 x 16-in. hollow block, containing two or three vertical cores, or from 30 to 40 per cent hollow space. This is the usual test specimen referred to. Most specifications require this unit to have a compressive strength, at 28 days, of 700 lbs. per sq. in. of gross area. It is well to bear this in mind when reading the following pages.

Before considering the various factors involved in making cinder concrete building units, we shall state the problem which

the industry is trying to solve. It is, briefly: To produce at the lowest possible cost a cinder concrete which combines cellular structure with the necessary structural strength. Strength, then, is not the sole criterion for quality—the cellular structure should be developed to the fullest possible degree.

Full understanding of the value of cinder concrete is not attained without realizing the peculiar character of the aggregate —its porosity. This porosity, and the corresponding limited strength, places certain limitations upon the use of the resulting concrete; but these limitations do not make the concrete inferior except when it is considered beyond its proper field of application. Within the field it has its distinct advantages.

While strength is not the sole criterion for quality, the required strength for the particular purpose must, of course, be met. As above stated, 700 lbs. per sq. in. is the usual requirement for hollow, load-bearing units. The manufacturer is primarily interested in obtaining this strength, and obtaining it economically. His first concern in his design of mix and in his method of making is that of obtaining the greatest possible yield per sack of cement for the strength necessary. Strength can be measured very simply—the other qualities residing in the cellular structure usually become secondary, but they should never be lost from sight.

Cinder Aggregate

The permissible maximum size is limited by the type of unit to be manufactured from the aggregate. The particles must be sufficiently strong to withstand the pressure or tamping—otherwise they are broken in the mold box of the machine, thereby creating new surfaces not coated with cement mortar. Generally speaking, the particle size should not exceed one-quarter to one-third the smallest dimension of the unit.

The cellular structure of cinder concrete is dependent upon the cells (sealed) and the pores (open) of the cinder aggregate. Of these, the sealed cells are particularly desirable, while the open, inter-connecting pores should be reduced to minimum size. The strength is dependent upon the strength of the particles and their bond. To get strength and proper porosity of the concrete,

grading of the aggregate is necessary. The unevenly shaped particles of the cinders contain numerous contact surfaces—these must be covered with mortar composed of cement and finer particles, and must be in contact. It is not desired, however, to fill the pores of the individual particles. Mortar lodged in these pores does not perform its function—on the contrary, it reduces cellularity.

The voids of a typical cinder aggregate, as used by the makers of masonry units, appear to approximate 55 per cent of the total volume of the loosely packed aggregate, when measured by the water displacement method. These "voids" represent not only the pores or pockets of the particles, but also open space between the particles. In making cinder concrete, such open space should be filled as completely as possible by cement mortar, to eliminate large channels through the mass. The pores or pockets of the particles are, to a large extent, filled with water which prevents mortar from getting in (unless the mixing process is *too* thorough). As far as the sealed and inaccessible cells are concerned, Nitzsche[11] found them to represent from 8 to 16 per cent of the total volume. These cells may be determined from the specific gravity of the finely ground material. A cinder aggregate should be graded from coarse to fine so the cement paste becomes most efficient. We are concerned with contact surfaces of rather irregularly shaped particles, and the problem is certainly less elementary than with stone concrete. The method of solving it is, of necessity, empirical.

The writer has conducted a large number of tests involving anthracite cinders; 8 x 8 x 16-in blocks, two-core, 30 per cent air space, 1 to 7 mix, were manufactured on a power press. All variables were controlled to give best results. The water content was varied according to porosity of aggregate, but was close to 10 gal. per sack of cement. For these particular conditions, a maximum strength cement ratio was obtained at a fineness modulus of 4.00 to 4.50 ($\frac{3}{4}$-in.-0). Other tests by manufacturers of cinder units have generally supported this, but conditions vary and it is difficult to evolve universal rules. More recent tests, using a highly clinkered bituminous cinder, point towards a lower fineness modulus and a lower maximum size, but the data are not conclusive. The common maximum size of the

cinder unit industry is probably less than ¾-in. A large variety of units are made with thin webs which necessitate smaller maximum particles.

Nitzsche conducted similar experiments, but does not report his grading in great detail. He prepared prisms which he tested for transverse strength. The results are quoted as follows:

Aggregate No.	A	B	C
Sizes, by volume:			
.47″—28″ (12 mm—7 mm).....	1 part	1.00 part	1.00 part
.28″—16″ (7 mm—4 mm).....	1.25 part	2.00 part	1.00 part
.16″—08″ (4 mm—2 mm).....	.18 part	.50 part	.50 part
.08″—00″ (2 mm—0 mm).....	.33 part	.25 part	2.00 part
Weight per cu. ft.....	60.6	56.8	68.8 lbs.
1:4.5 cinder concrete			
Weight per cu. ft.....	85.0	75.0	81.8 lbs.
Porosity by absorp. test.....	28.5%	27.2%	31.0%
Transverse strength lbs. per sq. in.....	2440	1720	1400
1:7.0 cinder concrete			
Weight per cu. ft.....	78.1	70.0	74.9 lbs.
Porosity by absorp. test.....	32.6%	32.2%	30.7%
Transverse strength lbs. per sq. in.....	1460	940	1170

Aggregate *A* is the best graded of the three and produces the strongest concrete. *B* contains too much material in the range from about ¼-in to 3/16-in. Its showing is particularly bad with the leaner mix (1:7) where the lack of fines becomes more pronounced in its effect. *C* contains too much fine material. In lean mixes these fines are of increasing importance. Note that this aggregate is rather heavy; evidently the weight is not a safe ndicator of proper grading.

If a general rule were to be given for the grading of cinder aggregate to produce masonry units of strength 700 lbs. per sq. in. for maximum yield per sack of cement, a fineness modulus of 4.0 to 4.5 (¾-in.-0) might be suggested. It is important, however, to remember that so many other variables enter the manufacture, that it becomes necessary for the maker of cinder units to run his own tests to obtain data applying to his conditions. The variables not only involve those of the raw material, but also the machine used and general method of making and curing.

Experience shows that the run of the grate of cinders from one reliable source, crushed by a uniform method to produce the desirable maximum size, is generally surprisingly uniform in its grading. If this were not the case, it would be impossible for cinder unit plants to produce a uniform product. Separation of cinders into definite sizes, by screening, is very difficult because of the nature of this damp, sticky material. From a practical point of view, (not losing sight of cost), it is perhaps better to endeavor to keep the cinders together, i. e., to prevent segregation. This segregation is a real difficulty in many plants, and it is readily multiplied when screening is used. The manufacturer, then, relies on a uniform and reliable source of supply

Scientific grading by separation and recombination is desirable and deserves considerable attention. The above grading is proposed, mainly, as guidance in empirical selection of the raw material. Considerable control may be obtained by proper operation of the crusher.

Much has been said of the effect of the fine particles of cinders. The writer refers to his definition of cinders to exclude household ashes. Cinders from high temperature combustion do not contain any appreciable amount of fine, fluffy ash—they contain some of fine gritty material. These fines are rarely present in any undesirable excess—they are needed to make the otherwise harsh mix workable. A typical cinder aggregate usually contains less than 10 per cent finer than No. 100 mesh.

The average manufacturer of cinder units unfortunately does not perform a large number of sieve analyses. Realizing that the raw material, as run of the grate, usually contains the full range of particles, the writer has proposed as a simple rule that the aggregate should "split" 50-50 or 40-60 on ¼-in. sieve. In other words, the manufacturer should endeavor to obtain such division through selection of supply and operation of crusher. It is admitted that freak aggregates can be produced without violating this rule of division on ¼-in.—but the writer has rarely encountered them except in patent litigation.

Mixing Water

When considering the amount of mixing water to be used in making cinder concrete building units, it must be borne in mind

(1) that we are here dealing with a highly absorptive aggregate and (2) that this concrete mixture cannot be considered as being truly plastic.

A large amount of water is usually present in the cinders when they arrive at the plant, or in the aggregate when it reaches the mixer. The manufacturer describes his aggregate as "dry," "wet" or "damp", but few realize how much moisture is actually contained. Part of the water which may be absorbed by a cinder aggregate is readily drained off—but a considerable amount remains even after prolonged drainage. The following data may be of interest:

A bituminous cinder aggregate was obtained from a plant in which the cinders were washed before crushing; they were therefore very wet in the storage bin. The fineness modulus was 3.85.

	Moisture	Weight Per Cu. Ft. Loose	Rodded
	%	Lbs.	Lbs.
Aggregate as received	38.7	57.0	62.6
Dry cinders in wet aggregate		41.1	45.2
Dried aggregate	0.0	54.0	61.7
Same soaked 15 min. and then drained of free water	31.4	57.2	64.0
Dry cinders in this aggregate		43.5	48.7
Same drained 18 hours	9.2	47.7	56.2
Dry cinders in this aggregate		43.6	51.4
The cinders were then inundated:			
Dry cinders per cu. ft. of inundated aggregate		50.0	
Water per cu. ft. of inundated aggregate		33.5	

The cinders of the above test were drained by being placed in a canvas bag. After 18 hours they still contained 9.2 per cent of water. The aggregate, when taken from the storage bin, contained close to 16 lbs. of water per cu. ft. Used in a 1:7 mix, this would mean more than 13 gallons per sack of cement. In this particular plant, where the cinders were washed, no water was added in the mixer when the aggregate under consideration was used.

From the above data it is possible to determine the bulking of the cinder aggregate, loose:

54 lbs. dry cinder aggregate (F. M. 3.85) ..... 1.00 cu. ft.
54 lbs. dry plus 9.2% water ..... 1.24 cu. ft.
54 lbs. dry plus 31.4% water ..... 1.24 cu. ft.
54 lbs. dry plus 38.7% water ..... 1.31 cu. ft.
54 lbs. dry plus 67.0% water (inundated) ..... 1.08 cu. ft.

It is apparent that considerable bulking takes place. For the percentages recorded, this bulking is approximately constant within the limits of moisture usually found in cinders. When inundated, the aggregate did not return to the volume of the dry aggregate, presumably because of the presence of particles of low specific gravity. A "pushing apart" may take place.

The data emphasize the necessity of using care in applying results obtained with dry cinders in the laboratory to wet cinders on the job.

While the above data were obtained from a plant using washed cinders, the writer has found the moisture content, in cinder unit plants using cinders as they come from the source, to range from 10 to 25 per cent, ignoring extremes. The percentages quoted here are based on dry weight.

It is evident from all figures that a large part of the total mixing water, with this aggregate, is present in the cinders and that this must be taken into account. The writer is aware of many experiments in which only the water added in the mixer was measured. Inasmuch as the mixing water is an important factor, here as in other concretes, such experiments cannot be accepted.

The amount of mixing water to be used to obtain maximum strength-cement ratio, depends upon the nature of the aggregate. The following series involves 8 x 8 x 16-in. hollow blocks made on a tamping machine. The carbon content of the cinders was varied—starting with a well clinkered bituminous cinder and adding carbon in the form of ¼ anthracite, ¼ bituminous coal, and ½ coke. It was difficult under these conditions to judge the consistency of the mix, but it is believed that "best results" were obtained in each case.

Mix 1:7 Dry Rodded. Coal and Coke in Cinders—Mixed as Above Stated	U. S. Gallons Water/Sack of Cement	U. S. Gallons Water/Cu. Ft. of Aggregate
20%	13.0	1.85
35%	11.9	1.70
50%	10.8	1.55
75%	10.2	1.45
100% (aggregate consisting of coal and coke only)	9.1	1.31
Mix 1:9 dry rodded. 20% carbon in cinders	16.7	1.85

The addition of dense coal reduces the amount of mixing water to be used. The density of the coal here more than offsets the effect of the porous coke.

The amount of mixing water to be used in making cinder concrete building units must be determined in every instance under the conditions prevailing in a plant. For example, a test run in an Eastern plant, using anthracite cinders (F. M. 4.0) in making 8 x 8 x 16-in. hollow blocks (30 per cent air space) on a power press, gave the following results for a 1:7 mix:

Mixing Water	Compressive Strength 28 Days
7.8 gallons per sack of cement	624 lbs. per sq. in.
9.6 gallons per sack of cement	741 lbs. per sq. in.
10.5 gallons per sack of cement	835 lbs. per sq. in.

A later test run at this plant, using a somethat coarser aggregate (F. M. 4.5) brought best results when using 12 gal. of water per sack cement—additional water giving slump of the block and decrease in strength.

It may be of interest to consider another test run involving two types of mixes. Anthracite cinders of fineness modulus 4.0 were used throughout, same cement, 7 min. mixing time, same type of 8 x 8 x 16-in. hollow block (30 per cent air space), same method of curing. Three different types of block machines were used, but in each instance the one variable of water content was controlled with a fair degree of accuracy to produce maximum strength for the machine in question.

1. Power press:
   1: 7 mix: 10.4 gal. water per sack of cement.
   1:10 mix: 15.9 gal. water per sack of cement.
2. Heavy tamping machine:
   1: 7 mix: 11.8 gal. water per sack of cement.
   1:10 mix: 15.5 gal. water per sack of cement.
3. Light tamping machine:
   1: 7 mix: 11.0 gal. water per sack of cement.
   1:10 mix: 14.3 gal. water per sack of cement.

If a general rule were to be deduced from these tests on anthracite cinder aggregate of fineness modulus 4.0, it would be that a 1:7 mix requires about 10 to 12 gal. of water per sack of cement, and that a 1:10 mix requires about 15 to 16 gal. In the last

mentioned series we may figure the amount of water used per cubic foot of aggregate, loosely packed:

1. 1: 7 mix: 1.49 gal. per cu. ft. of aggregate.
   1:10 mix: 1.59 gal. per cu. ft. of aggregate.
2. 1: 7 mix: 1.71 gal. per cu. ft. of aggregate.
   1:10 mix: 1.55 gal. per cu. ft. of aggregate.
3. 1: 7 mix: 1.57 gal. per cu. ft of aggregate.
   1:10 mix: 1.43 gal. per cu. ft. of aggregate.

Average: 1.56 gal. per cu. ft. of aggregate.

In the above described test on bituminous cinders of varying carbon content, it was found that the mixing water for the aggregate containing 20 per cent carbon (mainly coke) was the same in 1:7 and 1:10 mixes, viz., 1.85 gal. per cu. ft.

As previously pointed out, one-half or more of the total mixing water is generally present in the cinders as received from the original source. It is believed that it is desirable to have at least part of the water present in the aggregate when the cement is introduced—otherwise part of the binder may be lodged in the open pores of the particles.

In cinder unit plants, the amount of water to be added in the mixer is mainly a matter of judgment. Drainage of the aggregate in the storage bin causes considerable variation; a mix is used which cannot be judged by slump; it is thus for the mixer man to know. Frequent test runs are important. It is well to employ an automatic measuring tank for the water, even if it has to be adjusted several times a day; it does reduce a variable in the manufacture.

Method of Mixing

The paddle mixer with open stationary drum is the type most commonly used in cinder unit plants. The lightness of the aggregate and the harshness of the mixture favor this type of equipment.

Tests have shown that the length of mixing time has a marked influence upon the quality of cinder concrete. This is readily understood when considering the nature of the aggregate. The shape of the particles is such that the surface area is very large. The cement paste (mortar) must be distributed throughout. It is probable that the shape of the particles (and perhaps their lightness) also require thorough mixing in order to obtain the

desired distribution of all sizes throughout. Tests indicate that too prolonged mixing may lead past the desired maximum—probably because cement is forced into pores where it does not serve as binder.

The majority of tests on record indicate that an increase in strength of cinder concrete continues up to a mixing time of seven minutes. The following test is selected at random:

Aggregate: Anthracite cinders, fineness modulus 3.82.
Total mixing water: 86 lbs. per sack cement.
Proportions: 1 to 7 damp loose.
9 cu. ft. paddle mixer, speed of shaft, 20 R. P. M.
8 x 8 x 16-in. blocks, 30 per cent air space, made on power press and stored in air for 28 days to 1 year.
Mixing time: 2 minutes.
28 day strength: 676 lbs. per sq. in.
1 year strength: 838 lbs. per sq. in.
Mixing time: 7 minutes.
28 day strength: 835 lbs. per sq. in.
1 year strength: 950 lbs. per sq. in.

The speed of the mixer blades is naturally of importance. It usually varies from 18 to 25 R. P. M. in cinder unit plants. The writer has conducted tests with mixer speeds up to 48 R. P. M., but the results at this high speed were erratic due to difficulty in charging and discharging the mix as well as in keeping the materials in the mixer. For the type of mixer commonly used in these plants, a speed of 25 R. P. M. appears most satisfactory. This applies to several makes of mixers, apparently of about the same efficiency in mixing.

Method of Molding

It is not within the scope of this paper to discuss the large number of machines on the market. The dominating type in the cinder unit field today is the so-called "stripper" which ejects the unit either upward or downward after tamping or pressing in the mold box.

Naturally, the strength of the cinder concrete increases with increased packing, provided the particles of aggregate are of sufficient strength to avoid crushing in the mold box. The method of feeding is important. Generally, the unit (usually 8 in. high) is weaker in its lower part ("the pallet side") because of the

bridging effect of the rather harsh cinders. This is particularly true of units with narrow webs. Best results are obtained, apparently, when machines strip the unit downward, as this permits the web to be wider towards the bottom, reducing the bridging effect. Feeding the material in layers also helps (see report of Committee P-1 in the A. C. I. Proceedings 1928).

Method of Curing and Aging

The hydration of cement in cinder concrete proceeds as in other concretes. It is of importance here to consider the amount of water present in the freshly made concrete.

Because of the porosity of the cinder aggregate, a large water reserve is available for hydration of the cement, and because of the nature of the concrete, the water is slow in reaching the surface where it evaporates. We have previously quoted instances of machine-made units containing more than 10 gal. of water per sack of cement with a 1:7 mix and more than 15 gal. in case of a 1:10 mix.

This water reserve simplifies the curing method. The "green" units may well be stored at a temperature of 70 to 80 deg F. without danger of too rapid drying. As a matter of fact, it becomes a problem to get rid of the water if the units are to be ready for safe use at 28 days. The significance of "dry" units becomes apparent when considering the problem of volume changes.

In one series of tests, the writer found 8 x 8 x 16-in. cinder units to contain more than 2 lbs. of water per unit after 28 days storage in the laboratory. The drying is more rapid when the units are properly stored in the yard. It is of the greatest importance to have the yard covered with a shed in order to prevent wetting from rains. The general problem of piling, etc. is now being studied. Of course, too rapid drying is possible, due to exposure to sun or to certain atmospheric conditions. But this is a rare occurrence.

Manufacturing Methods Generally

While the mechanical operations of mixing, molding, curing and storing the cinder unit, closely resemble those of a typical concrete block plant, the manufacturer of cinder units is, in addition, confronted with the problem of turning raw cinders into

a suitable aggregate. We shall briefly consider this part of the manufacture.

A plant producing one million 8 x 8 x 16-in. hollow units will consume approximately 20,000 cu. yds. of raw cinders, depending upon the nature of the material. Inasmuch as the manufacture of the units is rarely synchronized with the supply of raw material (usually contracted for), it is necessary to provide reserve storage and facilities for handling. Ground storage and exposure to weather offer the advantage of getting the cinders washed by rain (weather permitting) more thoroughly than is possible by any mechanical washer. The content of soluble sulphates is reduced. Of course, drainage must be provided and the cinders must be drawn from the top of the pile; otherwise the aggregate may suffer from a concentration of soluble salts. While a clamshell bucket is very satisfactory for handling the cinders from the pile to the aggregate plant, it is generally too expensive to operate except in large plants; it requires a licensed operator. Drag line buckets have been used successfully, the main difficulty here being the wear on the cables sliding through the abrasive cinders. A system of portable belt conveyors is used in many plants. Others use a tractor with a scraper attachment.

The effort in cinder unit plants is to consume the cinders as soon as they arrive. They are usually delivered in gondola cars holding 40 to 80 cubic yards. In designing conveyors for cinders, raw or crushed, it must be borne in mind that the material is highly abrasive. Elevators should be slow moving of the continuous bucket type and the feeding so arranged that spilling is avoided as far as possible and that facilities are present for easy cleaning of the boot. Belt conveyors should be wide and well troughed and provided with skirt boards to prevent spilling.

The crushing method depends upon the nature of the cinders. Usually a primary crusher is employed for reduction to 1½ to 2 in., while a secondary crusher completes the reduction to the required aggregate size, say ¾ in. Jaw crushers and hammer mills are most frequently used as primary crushers. The former type serves well when there is not too much fine material present to cause clogging. When the latter type is used, the hammers should move at much less speed than for limestone. The grate

bars should be spaced about two inches. To attempt the complete reduction of raw cinders in a hammer mill (requiring high speed and close spacing) does not appear economical. Roll crushers are used for secondary reduction. A common type consists of two pairs of rolls, each of 16 in. diameter. If but one set of rolls is used, larger diameter is required to get the necessary nip. Of course, rolls may also be used in primary reduction if the rolls are properly spaced. The usual care must be used in obtaining the proper spring adjustment; perhaps this is particularly important here because of the difference in hardness of the particles of cinders. It is necessary to have the feed evenly distributed over the full width of the rolls—otherwise the wear becomes excessive in spots. Sometimes cinders are obtained of maximum size less than 2 in.; the reduction may then be obtained in one step, using a roll crusher.

The crusher units should always be operated in circuit with a screen. The crushers cannot be relied upon to control the maximum size of the aggregate, and the output should therefore be screened so that any oversize may be returned. It is not good practice to pass anything through a crusher that does not need crushing; the cinders should therefore first be screened. The two screening operations may be combined, at least in a medium size plant, by first elevating all cinders to a screen on top of the aggregate bin, letting the aggregate pass through and returning the oversize to the crusher, thereafter joining the raw cinders on their way to the screen. This type of closed circuit is most desirable; it saves the crushers, prevents undersirable breaking down of particles not in need of reduction, and insures against over-size getting into the storage bin. Of course, the layout of equipment of this type depends upon the nature of the raw material and the capacity of the plant.

Screening of cinders is, at the best, an incomplete operation. Cinders are always wet when they arrive at the plant, and fine particles adhere to the surface of, or are occluded in, the pores of coarser particles, and cannot be removed except after drying. The dampness of the cinders and the irregular shape of the particles cause blinding of even the best screen. It is not necessary, however, to obtain complete separation when the

object is merely to save the crusher from unnecessary work or to prevent too coarse particles from getting into the aggregate bin.

Revolving screens are very unsatisfactory for cinders. The screening efficiency is notoriously low, and blinding quickly takes place when damp cinders are introduced. Fast vibrating screens, mechanically or electrically agitated, produce the best results. The type of screen cloth to be used depends upon the nature of the raw material. Each case deserves expert attention.

One of the significant problems of a cinder unit plant is the prevention of segregation of the raw cinders and the cinder aggregate. This segregation is particularly pronounced when the aggregate is fed into a large bin from a bucket elevator. The thin stream discharged by the elevator segregates readily when striking the cone of aggregate built up in the bin. In several plants this is successfully overcome by having the elevator discharge into a dumping box of capacity ½ to 1 cu. yd. When the box is full, it dumps its entire contents into the bin, thereby avoiding the slow building up of the cone. It is further desirable to have the storage bins of small diameter (8 ft. or so), with steeply inclined bottom (about 45 degrees) leading to a central discharge opening. It is well to endeavor to keep the storage bin full.

Cinders, consisting of rough particles, light and damp, do not flow readily, and all chutes should be designed with a slope of at least 50 degrees. Otherwise segregation occurs.

MAKING SAND-CINDER CONCRETE

The writer has done little work with sand-cinder concrete, and must therefore rely, to a great extent, upon published data. Unfortunately, such data are incomplete. All that is generally reported are the arbitrary proportions and the compressive strength—with no information regarding the properties of the raw material, sand and cinders, or the design of mix (grading and water content), or the method of making. To know that a "1:2:5 sand-cinder concrete" developed a compressive strength of 800 lbs. per sq. in. is of no help to the maker of such material unless he knows more about its history. The wide variation in compressive strength reported certainly suggests the need of more data.

We are here concerned only with sand-cinder concrete as it is poured on the job, usually for floors and fireproofing. The problem is different from that of cinder concrete building units, where the concrete is machine made under pressure. Under such conditions, the harshness of the cinder concrete does not make it impossible to obtain uniformity. On the other hand, if the concrete is placed by hand, around reinforcing, it must flow more readily, and this is helped by the introduction of sand.

The main reason for the use of cinders in this type of concrete is the lightness of the aggregate. The main reason for the addition of sand is to produce workability. Strength and density are not the only criteria for quality in the field where sand-cinder concrete properly belongs. A certain strength is necessary, and sand is a helpful agent because conditions on the job cannot be controlled to the same extent as conditions in a plant. A certain density is necessary, but it is unfortunate that many investigators of sand-cinder concrete place density as a chief aim.

Cinder concrete may serve the purpose of sand-cinder concrete if the aggregate can be well enough controlled on the job and if the necessary care and effort are used in placing. The writer is familiar with several jobs of reinforced concrete (roofs, floors, beams) where a straight cinder aggregate was used—and large quantities of reinforced cinder concrete lintels are being produced by cinder unit plants, using consistencies ranging from "dry" to "wet."

The difficulty of rigid control of the cinders on the job suggests the use of sand as part aggregate. With a weight of 100-120 lbs. per cu. ft. (85-90 lbs. for cinder concrete) the sand-cinder concrete is still considerably lighter than stone concrete.

Of course, in addition to its lightness, sand-cinder concrete has other advantages inherent in the cellular cinders. These will be considered later.

Sand-Cinder Aggregate

The mixture of dense and cellular aggregates naturally possesses characteristics intermediate between the two extremes. What has been said about cinder aggregate will to some extent apply here.

Nitzsche[11] reports the following, in which he uses the cinder mixtures previously described under cinder aggregate, substituting for ¼ of the cinders, sand graded from about ¼ in. down.

Cinder *A:*

Sizes, by volume:

.47—.28 in. (12—7mm)	1 part
.28—.16 in. (7—4mm)	1.25 part
.16—.08 in. (4—2mm)	.18 part
.08—.00 in. (2—0mm)	.33 part

1 cement:4.5 cinders:	85.0 lbs. per cu. ft.
	28.5 per cent pores (by absorption test)
	2440 lbs. per sq. in., transverse strength.
1 cement:3.4 cinders:	
:1.1 sand	95.5 lbs. per cu. ft.
	26.3 per cent pores
	2140 lbs. per sq. in., transverse strength.
1 cement:7.0 cinders:	78.1 lbs. per cu. ft.
	32.6 per cent pores
	1460 lbs. per sq. in., transverse strength.
1 cement:5.3 cinders:	
:1.7 sand	88.1 lbs. per cu. ft.
	28.7 per cent pores
	1310 lbs. per sq. in., transverse strength.

The cinder aggregate appears to be well graded. Substituting sand for ¼ of these cinders does not improve this concrete; the weight increases (the sand being heavier)—the porosity decreases (the sand being denser)—but the compressive strength falls off. The reduction in transverse strength is more noticeable in the richer mix.

Cinder *B:*

Sizes, by volume:

.47—.28 in. (12—7mm)	1.00 part
.28—.16 in. (7—4mm)	2.00 part
.16—.08 in. (4—2mm)	.50 part
.08—.00 in. (2—0mm)	.25 part

1 cement:4.5 cinders:	75.0 lbs. per cu. ft.
	27.2 per cent porosity (by absorption test)
	1720 lbs. per sq. in., transverse strength.
1 cement:3.4 cinders	
:1.1 sand	96.2 lbs. per cu. ft.
	24.4 per cent porosity
	2820 lbs. per sq. in., transverse strength.

1 cement:7.0 cinders:	70.0 lbs. per cu. ft.
	32.2 per cent porosity
	940 lbs. per sq. in., transverse strength.
1 cement:5.3 cinders	
:1.7 sand:	87.5 lbs. per cu. ft.
	26.1 per cent porosity
	1770 lbs. per sq. in., transverse strength.

This cinder aggregate is not well graded; it lacks finer particles. The addition of sand is therefore beneficial, and the strength increases notably in the leaner mix.

Cinder *C:*

Sizes, by volume:

.47—.28 in. (12—7mm)	1.00 part
.28—.16 in. (7—4mm)	1.00 part
.16—.08 in. (4—2mm)	.50 part
.08—.00 in. (2—0mm)	2.00 part
1 cement:4.5 cinders:	81.8 lbs. per cu. ft.
	31.0 per cent porosity
	1400 lbs. per sq. in., transverse strength.
1 cement:3.4 cinders:	
:1.1 sand:	91.2 lbs. per cu. ft.
	27.3 per cent porosity
	1600 lbs. per sq. in., transverse strength.
1 cement:7.0 cinders:	74.9 lbs. per cu. ft.
	30.7 per cent porosity
	1170 lbs. per sq. in., transverse strength.
1 cement:5.3 cinders:	
:1.7 sand:	86.2 lbs. per cu. ft.
	27.8 per cent porosity
	1205 lbs. per sq. in., transverse strength.

This cinder aggregate contains too many fines. Substituting sand for ¼ of the aggregate (mostly fine) does not alter the strength much—it reduces the porosity and increases the weight.

It is apparent that in these tests the introduction of sand does not necessarily improve the strength of the concrete. Much may be accomplished with cinders alone, if the aggregate is well designed.

In their comprehensive paper on reinforced sand-cinder concrete, Perrine and Strehan[10] present some information regarding the aggregate used in tests. Cinders, and 8 x 16-in. cylinders of sand cinder concrete, were obtained from jobs in New York City.

The cinders (anthracite) were described as follows:

A: Cinders from apartment house zone (1914).
B: Cinders from zone between apartment house section and downtown.
C: Cinders from downtown section.
D: Cinders from Columbia University power plant.

The sand was the typical bank material obtained from the north shore of Long Island.

The grading of the cinders and sand was as follows:

	Percentage of Cinders Coarser than Sieve No.			
Sieve No.	A	B	C	D
1¼ in.	2.2	5.6	1.9	0.0
1 in.	2.8		3.7	5.7
¾ in.	6.6	19.7	11.7	9.7
½ in.	14.1	42.7	35.5	19.5
¼ in.	36.6	75.7	74.6	26.1
10	61.2	88.3	88.9	84.9
16				90.5
30	75.6	90.7	97.0	94.9
50	78.4	93.8	98.7	96.6
100	86.3	96.4	99.5	97.7
Estimated F. M. standard scale	4.29	6.02	6.00	5.14
F. M. of sand (approximate)	2.37	2.72	2.64	2.66

Combining cinders and sand for a 1:2:5 mix should produce the following fineness moduli of the combined aggregates:

	Fineness Modulus		
	Sand	Cinders	2:5 Mixture
A	2.37	4.29	3.75
B	2.72	6.02	5.08
C	2.64	6.00	5.04
D	2.66	5.14	4.43

In making (on the job) concrete specimens (8 x 16-in. cylinders) from these aggregates, the mixer used and the brand of cement were not the same, nor is there any record of the amount of mixing water. The results are not entirely comparable—yet it may be of some interest.

	A	B	C	D
F. M. of cinders	4.29	6.02	6.00	5.14
F. M. of 2:5 sand-cinders	3.75	5.08	5.04	4.43
1:2:5 sand-cinder concrete:				
Weight per cu. ft.	107	107	109	113 lbs.
Compressive strength lbs. per sq. in.:				
One month	407	818	980	787
Six months	933	1744	1478	
One Year	913	1465	1475	

Cinders *B* and *C* are particularly coarse, with about 75 per cent retained on ¼ in. The addition of sand should materially improve such aggregates. Cinder *A* contains a large percentage of fine material; it would have been interesting to know what kind of cinder concrete it would have produced; mixed with sand it did not make a very good concrete. Cinder *D* is rich in medium sizes (¼ in. to No. 10), and the addition of sand does not bring it quite on par with *B* and *C*

If it were possible to draw a definite conclusion from these tests (if other variables were really controlled), it would favor a fineness modulus of the combined aggregates of about 4.5 to 5.0. Taken together with the previous test by Nitzsche, it again appears that blindly adding sand to cinders does not necessarily mean good or better concrete.

The lack of fines in the above cinders, as compared to the aggregate discussed under cinder concrete building units, is partly to be explained by the fact that these cinders were not crushed (beyond a breaking of large lumps with shovels), and partly to be explained by the usual wetness of New York cinders which may lead to actual washing away of fines during transport. It is also probable that the samples of aggregate were not taken with a view towards eliminating segregation.

Perrine and Strehan report tests on *D* with the cinders finer than ¼ in. "screened out." These tests show that the screening resulted in a decrease in strength of 1:2:4, 1:2:5 and 1:2:6 sand-cinder concrete as compared to same with the cinders unscreened. It is not stated, however, whether the screening was done while the cinders were wet, nor how efficient it was. Knowing the difficulty of screening cinders, it is impossible for the writer to draw any conclusions without knowing more how the screening was performed.

Richard L. Humphrey,[3] and Humphrey and Losse,[4] made tests on sand-cinder concrete beams at St. Louis in which considerable care was used in making the concrete. A product of excellent quality and remarkable uniformity was obtained, and the data are illuminating.

In the first series of tests, cinders were obtained from a power house of the United Railway Co. of St. Louis; this was presumably

a bituminous cinder. Weight, 47.0 lbs. per cu. ft. The sieve analysis was as follows, maximum size 1¼ in.:

Retained on	¾ in.	½ in.	¼ in.	No. 10	No. 20	No. 30	No. 50	No. 100
Percentage	18.6	39.7	63.1	78.9	86.3	89.5	93.5	95.8

Fineness Modulus of cinders: 5.65. Fineness Modulus of Meramac River sand: 2.42.

The mixtures were proportioned by weight to obtain a 1:2:4 mix of dry loose aggregate. Due to an error in determining the moisture content of the cinders (it proved to be 23 per cent), the actual mixes turned out to be close to 1:2:5. A 1 cu. yd. mixer was used, and the materials were mixed 2 minutes dry and 3 minutes wet. Three consistencies were used, described as "Wet," "Medium" and "Damp." In each the total water was measured and expressed (including moisture in aggregate) in percentage of weight of dry mixed materials. Cement was here assumed to weigh 100 lbs. per cu. ft. In each case a number of 8 x 16-in. cylinders were prepared, the following data having reference to these cylinders:

Proportions By Volume	Proportions By Weight	Water % of Dry	Weight of Concrete at 13 Weeks	28 Days Strength Lbs. Per Sq. In.
"Wet Consistency"				
1:2:5.06	1:2.02:2.38	21.6		1,243
same	same	21.8	116.5 lbs.	1,000
same	same	21.6		1,000
			Average:	1,081
"Medium Consistency"				
1:2:5.06	1:2.02:2.38	19.4		1,114
1:2.06:5.40	1:2 07:2.54	20.4	115.6 lbs.	1,250
same	same	20.4		1,240
			Average:	1,201
"Damp consistency"				
1:2:5.19	1:2.01:2.44	19.0		1,055
same	same	same	112.5 lbs.	1,210
same	same	same		1,090
			Average:	1.118

The average compressive strength of these cylinders at 6 months may be reported as follows:

	Weight Per Cu. Ft.	Compressive Strength Lbs. Per Sq. In.
Wet consistency	113.9 lbs.	2,021
Medium consistency	114.3 lbs.	2,203
Damp consistency	113.7 lbs.	1,945

In the second series of tests, cinders were obtained from the same source. With a maximum size of 1¼ in., the grading was:

Retained on....	¾ in.	½ in.	¼ in.	No. 10	No. 20	No. 30	No. 50	No. 100
Percentage.....	6.8	22.9	78.4	85.8	90.9	92.9	95.1	96.8

Fineness modulus: 5.69.

In this series of tests, sand-cinder concrete was prepared in the same manner as before, but the mixes were actually designed to be 1:2:4 by volume. This corresponded to about 1:2:2 by weight, the dry sand weighing about 100 lbs. and the dry cinders 50 lbs. per cu. ft. The total mixing water averaged 19 per cent of the dry materials for "medium consistency." Cylinders (8 x 16 in.) were prepared as before, and averaged as follows:

	Weight Per Cu. Ft.		Compressive Strength Lbs. Per Sq. In.
28 days	118.4 lbs.	High 1964 Low 1499 Average	1,647
13 weeks	119.4 lbs.		2,217
6 months	119.5 lbs.		2,525
1 year	119.7 lbs.		2,761

When observing the sand-cinder aggregate used in these tests it is interesting to find, first, how closely alike the cinders of the two series are. They were from one source, evidently producing a good grade of cinders. The fineness moduli are almost the same. Characteristic for both is a lack of fine material. Combining sand and cinders in the two general proportions produces the following:

	Fineness Modulus		
	Sand	Cinders	Mixture
1:2:5	2.42	5.65	4.72
1:2:4	2.42	5.69	4.60

The sand-cinder concrete of these tests was apparently of good quality. The strength for the 1:2:5 mixture is above those re-

ported by Perrine and Strehan, as representing "job made" New York concrete. It is unfortunately impossible to judge whether the sand cinder proportion was the most desirable for maximum strength/cement ratio, and it is also impossible to judge the density and other characteristics of the concrete. The many test specimens (representing the large number of reinforced beams prepared) show a most remarkable uniformity of strength. It is evidently quite possible to produce a uniform sand-cinder concrete if care is used in proportioning and if the cinders are from a reliable source. Other features of these tests will be considered later.

Some information regarding the effect of sand upon sand-cinder mixtures may be gained from tests made by C. M. Spofford and H. W. Hayward, as reported by Taylor and Thompson, "Concrete, Plain and Reinforced," 1916 edition, p. 328. We shall here omit the reference to the appearance of the cinders and the nature of the coal.

1:2:5 Sand-Cinder Concrete. 8 x 8 x 16-in. Prisms

% of Cinders Passing				Weight in Lbs. Per	28 Days Strength Lbs. Per
1 in.	¾ in.	½ in.	¼ in.	Cu. Ft.	Sq. In.
100	96	84	71	89	400
100	93	78	65	95	492
100	93	76	60	91	645
100	89	63	43	97	812
100	81	70	50	104	828
100	84	57	36	101	868
100	80	49	33	109	883
100	66	53	26	113	1088
100	83	47	26	111	1246
½-in. stone and coarse sand				145	1620

Different cinders were used in making these specimens and it is not known to what extent that affected the results. Nevertheless, it appears evident that there is a very definite relation between the grading and the strength, the latter increasing as the cinder-fines decrease from 71 to 26 per cent passing ¼ in. (the material through ½ in. decreasing in parallel manner from 84 to 71 per cent). It would be interesting to know how some of these mixtures would have behaved if no sand had been added. To add $^2/_5$

sand (by volume) to a cinder already having 71 per cent finer than ¼ in. will, of course, produce an altogether too fine aggregate.

Sand-cinder aggregate may be designed to produce definite results. It may not be possible to obtain the same degree of accuracy as with more expensive and better controlled aggregates, but an attempt should be well worth while.

Mixing and Placing

The purpose for which the sand-cinder concrete here considered is used, makes it necessary to obtain a mixture of greater plasticity than is required in cinder unit plants. It must be possible to work it around the wire mesh and rods of the floor slab, and distribute it well in the forms around the steel. On the other hand, it must not be so wet that the heavy sand and the light cinders segregate. There are few data available as to the effect of mixing water, mixing time or mixing methods on such concrete.

Perrine and Strehan state (page 580):

Cinder concrete should be mechanically mixed, for not less than 2 min., not leaner than 1:2:5, giving a minimum compressive strength, with standard cylinders at 28 days, of 800 lbs. per sq. in., and be in a wet to viscous consistency when cast.

Describing the making of floor slabs in the Columbia University laboratory, they state (page 550):

The process of mixing occupied about 2 min. per batch. The concrete was taken to the forms in wheel barrows, and when deposited was of a wet, viscous consistency. It was stirred and pushed into the spaces between the wires or rods with narrow pieces of scantling, the material being too wet to permit tamping, as the term is generally understood. An excellent, dense homogeneous concrete was thus obtained.

Note: A ¼-cu. yd. Blystone mixer was used.

As previously mentioned, Humphrey and Losse actually report the amount of mixing water used. Accepting the statement that "the water is figured in percentage of total weight of dry materials" to mean that the percentages are based on the weight of cement plus aggregates, the amount of water may be readily figured. Humphrey describes what is meant by "wet," "medium" and "damp" consistency, but it is not clear whether his descrip-

tion applies equally well to granite, gravel, limestone and sand-cinder concrete of his investigation.

It is apparent, immediately, that there is not much difference between the amounts of water used in producing the three consistencies. Apparently, the sand-cinder concrete mixture is just as sensitive to a pound of water at the point of proper consistency as other concrete mixtures. This is in accordance with the writer's observations of wet (almost plastic) mixtures of cinder concrete.

In the first series (1908) the mixing water (including absorbed moisture in the aggregates) is reported to be (in U. S. gal.).

1:2:5.06 sand-cinder concrete
"Wet" consistency:
14.1 gal. per sack cement.
1:2:5.06 sand-cinder concrete
"Medium" consistency:
12.6 gal. per sack cement.
1:2.06:5.40 sand-cinder concrete
"Medium" consistency:
13.7 gal. per sack cement.
1:2:5.19 sand cinders concrete
"Damp" consistency:
12.4 gal. per sack cement.

In the second (1911) series, the mixing water was about:
1:2:4 sand-cinder concrete:
11.4 gal. per sack cement

On the average, the amount of water for the 1:2:5 mix is about the same as the water used in a 1:7 cinder concrete mix as used in the manufacture of cinder concrete building units. The sand-cinder concrete, with the dense sand as part of the aggregate, has, of course, a consistency quite different from that obtained with straight cinder aggregate.

In the tests by Humphrey and Losse it is apparent that, for the particular cinder used, the amount of mixing water (including the large amount of moisture in the aggregates) should be about 13 to 14 gallons per sack of cement for a 1:2:5 mix, and about 11 to 12 gallons per sack of cement for a 1:2:4 mix. The mixing time used here (5 minutes total) should be sufficient.

Most authors agree that the sand-cinder concrete mixture must be wet. It can unquestionably be too wet. The test by

Humphrey indicates that (see above). The temptation of adding altogether too much water to stone concrete is well known—the same temptation is present here, perhaps to a greater extent. The cinders make the mixture harsh, yet the mixture can be properly placed without approaching the "liquid" form. The writer has observed that for reinforced cinder concrete, with no sand, carefully placed in cinder unit plants where the workmen knew a bit about the material they were handling, the sand-cinder concrete can be made so wet that it flows without any effort on part of the workman, but there is then great danger of segregation of the two aggregates, sand and cinders, because of the different specific gravities.

Stone concrete is gradually getting on a basis of rational design. The writer can see no reason why sand-cinder concrete should not follow. To specify arbitrarily a mixture of 1:2:5 just because one of the aggregates is not known to the second decimal is rather absurd. The engineer should know a little about his aggregate beyond the fact that it is "cinders," and he should run regular trial batches to determine just what he can accomplish with the material at hand.

On the Making of Sand-Cinder Concrete Generally

It is more difficult here to watch the cinders than in a cinder unit plant which usually contracts for a year-round supply. The storing of cinders on the job is usually limited to small quantities; generally the material is conveyed directly from the truck. The problem, however, is the same as in the cinder unit plant, viz., to prevent segregation.

The difficulty (if not impossibility) of crushing and screening on the job makes it necessary to obtain a cinder that does not contain much in excess of 1 in., or which is readily broken by shovel or in the mixer. The anthracite cinder of New York is of this type, as indicated by tests quoted.

Most of the data here quoted deal with 1:2:5 sand-cinder concrete, the apparently typical New York mixture. While it cannot be judged without knowing the nature and grading of the cinders to which the sand is added, it must be considered as being, generally, too lean. Attention should be directed to richer

mixtures, and to the proper gradation of the combined aggregates. A too harsh sand-cinder concrete leads to the use of too much water to get it to flow into place—with the result that the cement paste is diluted and segregation is likely to occur. The deficiencies of 1:2:5 mixtures should not be considered a reflection upon better designed mixtures of sand-cinder concrete.

The writer is convinced that there should be a considerable market for a prepared cinder, crushed and graded to meet definite requirements. This is probably the only way in which rationally designed cinder or sand-cinder concrete mixtures may be obtained. The chief obstacle is perhaps the fact that the contractor, faced with a "1:2:5" specification does not care what the "5" represents. Until this is remedied, sand-cinder concrete will not emerge from its present twilight zone.

Note on Job-Made Cinder Concrete

The foregoing pages may give the impression that job-made concrete (structural concrete), utilizing cinder aggregate, must of necessity contain sand. The writer has discussed sand-cinder concrete for this purpose, mainly because this type of concrete has been so generally used. He has pointed out, however, that the admixture of sand does not always improve the quality of the concrete—it all depends upon the nature (the design) of the cinder aggregate.

Since this report was first written, the writer has become familiar with some of the data obtained by the Thompson & Lichtner Co., Boston, in their work on structural, straight-cinder concrete. While these data (furnished by Miles N. Clair) are not incorporated in the present paper, it is hoped that they will be available in discussion of this report.

Cinder concrete mixtures of well defined slump were here produced. Small difference was found in the workability of cinders and other aggregates if they were all of the same grading. This applies particularly to mixtures 1:2:4 or richer. The data suggest as the most desirable a straight line grading on the semi-log plot with an allowable percentage of fines somewhat greater than is usual for sand. Ten per cent are permitted finer than No. 100 mesh. For usual maximum size (¾ in.) about 50 per cent should be retained on No. 4 mesh.

CHARACTERISTICS OF CINDER AND SAND-CINDER CONCRETE

In the preceding discussion of cinders it has been pointed out that the physical character of this aggregate is quite different from that of such aggregates as sand or stone, and that the resulting concrete should not be measured by the usual standards of strength and density. The field in which cinder, or sand-cinder concrete is used should be distinguished from the general field of stone concrete. The cinders should not be considered a substitute material--they produce a concrete possessing some highly desirable characteristics. These characteristics will briefly be reviewed here in the general discussion of the material.

Strength

First, concerning cinder concrete.

The effect of raw material and method of making upon the resulting concrete have already been discussed. The compressive strength will, of course, always depend upon the size and shape of test specimen, moisture content, etc. Hollow cinder concrete building units are usually made for a compressive strength of not less than 700 lbs. per sq. in. of gross area at 28 days. Because of many variables in the present day manufacture, it is necessary to aim at about 900 lbs. in order to be well above the minimum.

The compressive strength decreases with increasing moisture content. One series of tests on 8 x 8 x 16-in. hollow block of average compressive strength 827 lbs. per sq. in. showed a decrease in strength of 12 per cent after 48 hours immersion in water (tested wet). Another series of tests on brick of average compressive strength 4920 lbs. per sq. in. (tested flatwise) showed a decrease of 36.7 per cent after 48 hours immersion. The brick were denser than the block—evidently the effect of the water is more pronounced with lesser porosity.

Accepting cinders as an inert aggregate, it is natural to expect an increase in strength with age. Such increase is evidenced by many tests. When subjected to freezing and thawing treatments (described later), cinder concrete generally shows an increase in strength after completion of the test, obviously due to increased hydration of the cement during the submersion period.

There are few tests on record to show the strength of walls constructed of cinder concrete masonry units. They indicate a high wall efficiency (54.7 to 75.8 per cent), which is to be expected when considering the surface texture of machine-made cinder concrete. These same tests (on hollow cinder block piers, 54 in. high by 24 in. wide by 8 in. deep, laid in 1:3 cement mortar) showed a modulus of elasticity of the pier of 698,000 lbs. per sq. in. based on the intensity of stress on the gross cross section, and 1,035,000 lbs. per sq. in. based on the net section of 130 sq. in.

The transverse strength of cinder concrete building units is of interest chiefly in brick shapes, the larger unit rarely being exposed to tensile stresses in the wall. The transverse strength of cinder concrete brick increases with the compressive strength as indicated by the following representative tests: Cinder brick—standard size.

Compressive Strength	Modulus of Rupture
1221 lbs. per sq. in.	379 lbs. per sq. in.
1890 lbs. per sq. in.	496 lbs. per sq. in.
3035 lbs. per sq. in.	581 lbs. per sq. in.
4920 lbs. per sq. in.	765 lbs. per sq. in.

Second, concerning sand-cinder concrete.

It is not within the scope of this report to consider the design and behavior of reinforced sand-cinder concrete. The reader is referred to the papers by Humphrey, Guy B. Waite and Perrine and Strehan.

As to the concrete itself, Perrine and Strehan observe that:

> The crushing strength of standard (8 x 16″) cylinders of good cinder concrete (sand-cinder) as used in New York is between 800 and 1,000 lbs. per sq. in. at 28 days.
>
> The modulus of elasticity, E, under working loads, determined from standard cylinders, is about 1,200,000 lbs. per sq. in. for average concrete at 28 days.

As previously pointed out, Humphrey produced a considerably better sand-cinder concrete in his tests than the New York job concrete cited above. His 1:2:5 concrete ranged well above 1,000 lbs. at 28 days (8 x 16-in. cylinders), and his 1:2:4 concrete averaged 1647 lbs. at 28 days and reached an average of 2761 lbs. at 1 year. In the latter series of tests he produced 1:2:4 sand-gravel concrete of 3175 lbs. per sq. in., 1:2:4 sand-limestone con-

crete of 2492 lbs. per sq. in., and 1:2:4 sand-granite concrete of 3054 lbs. per sq. in.,at 28 days.

Humphrey and Losse report the initial modulus of elasticity (second series) to be:

1:2:4 sand-cinder concrete: 21 8 x 16-in. cylinders: 28 days:

High: 2,320,000 lbs. per sq. in.
Low: 1,180,000 lbs. per sq. In.
Average: 1,610,000 lbs. per sq. in.

For the 1:2:5 mixture, medium consistency, he reports the following initial modulus of elasticity:

At 28 days	Average: 1,352,000 lbs. per sq. in.
At 6 months	Average: 1,618,000 lbs. per sq. in.

Weight

The weight of cinder and sand-cinder concrete depends on the nature of the aggregate, design of mix, method of making, moisture content of the concrete.

A recent survey of the cinder concrete building units made by 28 plants gave the following results, 3 units from each plant being tested:

Weight of dry cinder concrete per cu. ft.

High value: 98 lbs.
Low value: 74 lbs.
Average: 87 lbs.

In one plant, the cinder concrete was found to weigh 106 lbs. per cu. ft., but this was due to an exceptional cinder, very high in iron. The above variation is wide—but these plants use widely different methods of making, in addition to representing cinders from all over the United States.

Perrine and Strehan report weights per cu. ft. of 1:2:5 sand-cinder concrete (using New York anthracite) ranging from 107 to 115 lbs. Humphrey reports his 1:2:5 sand-cinder concrete (using, presumably, a bituminous cinder) to weigh from about 112 lbs. to about 117 lbs. per cu. ft., while the average for his 1:2:4 sand-cinder concrete is 118.4 lbs.

Absorption and Allied Phenomena

It is still common belief that the old fashioned absorption test, in which the water absorption is determined by submerging the

specimen in water for a certain length of time, and is calculated in percentage of the dry weight, is indicative of such qualities as resistance to moisture penetration, frost action, etc. It would appear obvious, however, that this absorption test is of value only when it is representative of the actual conditions to which the material in question may be exposed. It may be of value with materials to be used under water—but its value is not clear for materials used in ordinary wall construction.

In the present paper, cinder concrete is considered as an intentionally porous concrete—in other words, a concrete utilizing a cellular aggregate in order to obtain certain characteristics—and not a concrete merely using cinders because they are cheap or because they are light. The value of the usual absorption test is questioned in a field where this porous concrete has proven its place. Cinder concrete does not belong where great density is needed. This may appear obvious, although it is not yet generally realized.

The immersion test gives no information regarding the behavior of cinder concrete in the wall of a building. Capillary attraction is the force which usually moves moisture through a wall, but that force is not measured by the standard test. Frost action depends upon the distribution of moisture throughout the wall, and upon the nature and distribution of the pores of the material, and perhaps upon other factors, but the immersion test gives no information about these matters.

It may be mentioned in passing that it is still quite common to specify an absorption limit for any kind of concrete in percentage of dry weight. How meaningless this is when including lightweight concrete is obvious. Assuming the limit to be 10 per cent by weight, 1 cu. ft. of stone concrete (140 lbs.) is permitted to absorb 14-lbs. of water, while 1-cu. ft. of cinder concrete (85 lbs.) must not absorb more than 8½ lbs. This has been remedied in most modern building codes; if a limit is considered necessary, it should probably be stated in lbs. of water absorbed per cu. ft. of concrete.

Frost action is usually determined in the laboratory by saturation of the material in question followed by alternated freezings and thawings. Such a test on cinder concrete building units is

reported and discussed in detail in the 1928 *Proceedings* of the American Concrete Institute. The freezing test specimens were soaked in water, then placed in the freezing chamber until frozen solid. After freezing for a period of 5 hours or more, during which time the temperature in the freezer was reduced to an average low value of 21 deg. F., the blocks were moved to a tank and thawed in water at 140 deg. F. for one hour. The blocks were cooled in water and put back into the freezer in the saturated condition, and the freezing and thawing was continued in like manner until 100 reversals had been made. The results are summarized as follows:

8 x 8 x 16-in. hollow cinder block.
Average compressive strength at 28 days (3 units): 1030 lbs. per sq. in.
Average dry weight at 28 days (3 units): 32.08 lbs.
Average absorption, percentage by weight at 28 days (3 units) 20.35 per cent.
Blocks normally cured, tested at 109 days:
Average compressive strength (3 units): 1042 lbs. per sq. in.
Average weight (3 units): 31.80 lbs.
Average absorption, percentage by weight (3 units) 20.12 per cent.
Blocks submitted to 100 freezing reversals, tested at 109 days:
Average compressive strength (6 units): 1123 lbs. per sq. in.
Average dry weight before freezing (6 units): 32.18 lbs.
Average dry weight after freezing (6 units): 31.75 lbs.
Average absorption, before freezing (6 units): 19.83 per cent.
Average absorption, after freezing (6 units): 23.12 per cent.

It will be seen that the blocks subjected to 100 freezing reversals showed an average compressive strength which was 7.8 per cent greater than the strength of the blocks normally cured. The absorption increased 16.6 per cent of the value before the freezing reversals, and the weight decreased 1.34 per cent. The increase in strength may be considered due to the warm water thawing treatment; the increase in absorption must be due to the opening, by the force of the freezing water, of cells which were previously inaccessible to water, as well as to other changes in pore structure. The significant facts are that the loss in weight was only 1.34 per cent and that the blocks showed no visible sign of cracking or spalling. No difference could be noted between the fractures from the compressive strength test of the frozen blocks and the blocks not frozen.

The writer has supervised several freezing and thawing tests in the last ten years. Qualitatively, the results were usually the same. An exception, however, is of interest. It involved cinder brick, and may be compared with another cinder brick made by the same plant, but from a different mix. The tests were carried out at Rochester, and at New York respectively. The results are summarized as follows:

	"Rochester" Brick	"New York" Brick
Freezing reversals	42	20
Compressive strength of brick not frozen	3290 lbs. per sq. in.	4920 lbs. per sq. in.
Absorption, by weight	15.8%	6.34%
Absorption corrected for weight according to A. C. I.	11.5%	4.65%
Change in strength due to freezing reversals	2.2% increase	24.1% decrease

Based on the recognized tests for strength and density, the "New York" brick is far superior to the "Rochester" brick, but the recognized freezing and thawing test showed the Rochester brick to be far superior. This may be compared to another test, made in 1922. The writer prepared cinder brick (using the same machine as above) from cinders crushed and screened to pass ⅛-in. mesh. At 28 days the brick tested 3,035 lbs. per sq. in. average, and had an absorption of 10 per cent—thus passing most quality requirements. When subjected to freezing and thawing test, however, they completely disintegrated within 20 reversals.

It would appear from this that with cinder concrete the absorption test does not give any information regarding the resistance to frost action, unless, perhaps, a minimum requirement should be considered. 15-20 per cent absorption by weight appears to be desirable. The action in freezing tests may be explained in two ways. It may be assumed that a highly cellular cinder concrete cannot be completely saturated with water, but that imprisoned air in the open pores (or the closed cells when forced open) permits expansion of freezing water without danger of fracture or disintegration. It may also be assumed that porous cinder concrete, in a standard freezing test, permits the water to drain off so rapidly that the concrete is far from being saturated when placed in the freezing tank. This may be taken to imply

that the standard test should be changed to insure complete saturation at all times—but would that be more truly representative of actual conditions? A material that drains rapidly in the wall, should also be permitted to drain rapidly in a test designed to determine its "wall behavior."

We shall next consider moisture movement through the wall. The usual absorption test is supposed to tell whether or not a material will produce a dry wall. Surely, that supposition is not justified unless we know something about the manner in which moisture travels through the wall. It is safe to assume that 100 per cent density will keep moisture out and that 100 per cent porosity will let moisture pass, but the fixing of a limit between these two extremes is not a matter of round-table discussion.

Suction is an important factor. A driving rainstorm does not ordinarily penetrate an 8-in. wall by its own force; the water is usually drawn by capillary force. It has been claimed for cinder concrete that this capillary force is very low because of the widening out of open channels into air pockets created by the porous cinder aggregate. The dependancy upon the porosity of the cinder concrete has not yet been proven, although most cinder concrete building units appear to have a rather low capillary attraction for water. Tests on cinder concrete and sand-cinder concrete indicate that the shape and distribution of the pores are the important factors, and that these cannot be judged by simple tests for porosity or absorption.

The moisture movement in a wall also involves the rate of drying. It is important that the wall should dry readily after getting wet, otherwise a magazine of water may be built up in the wall, leading to saturation. Of course, the drainage in a porous material helps in removing the water. There is apparently no definite connection between the rate of suction and the rate of drying; the distribution of the fine pores may be of chief importance. The usual absorption test gives no guidance.

The absorption test, or a simple determination of porosity, apparently gives no information regarding the true significance of the cellular structure of cinder concrete. We have previously shown (when discussing cinder aggregate) that there is no relation between porosity (determined by water absorption) and com-

pressive strength. This is also apparent from a number of tests conducted by the writer. The cellular structure, however, is a most important feature of cinder concrete—its chief characteristic —and should be first on the research program of the industry.

The introduction of a denser aggregate in sand-cinder concrete naturally affects the cellular structure. Writers on the subject have frequently indicated that it may be destroyed completely, and have placed the density of stone concrete as the goal for sand-cinder concrete. It is this, more than anything else, which has given cinders the appearance of being a substitute aggregate striving for something it cannot reach. A realization of the true nature of cinders should bring about an understanding of just what this aggregate will accomplish. As previously pointed out, the chief reason for the introduction of sand with cinders in the making of poured concrete is that of workability under prevailing conditions; it is not that of imitating stone concrete in other respects.

Volume Changes

The writer has previously mentioned the expansional movements which may be caused by certain types of coal or by sulphur compounds present in the cinders. It was pointed out that this phenomenon is rarely observed when the cinders come within the definition and specification of this paper.

Any concrete, however, is subject to shrinkage movements due to the hydration of the cement and to drying. Changes in moisture content cause changes in volume. Reference is here made to the paper by W. D. M. Allan, in this Journal (April 1930, Proceedings, Vol. 26, p. 699). It reports on sand concrete as well as cinder concrete. Sand-cinder concrete is perhaps intermediate in its behavior.

Part of the conclusions are quoted:

The use of units which are only partially dry, containing 2 to 5 per cent of moisture by weight, will result in the major amount of shrinkage which will occur in a concrete masonry wall.

For the conditions of air curing used in these studies, 26 days of air curing was generally sufficient to obtain blocks that would result in the minimum amount of shrinkage when laid in the wall.

In his foregoing discussion of manufacture of cinder units, the writer pointed out that the units must be piled in such a manner

that drying is facilitated, and that they must be protected against the weather.

These precautions do not apply to sand-cinder concrete, poured in situ. When properly reinforced it is unquestionably less subject to volume changes, or the evidence of such change does not become apparent. The design of the mixture is presumably of particular importance, but the data on hand do not justify any conclusions.

Fire Resistance

There is considerable evidence, and a great deal of opinion, available regarding the fire resistance of cinder and sand-cinder concrete. The majority seems to favor the material as being quite remarkable in this respect. A dissenting opinion, however, was voiced by the A. C. I. Committee E-4, in the 1929 Proceedings of the Institute (Vol. 25, p. 812). In summarizing its report, the committee concludes with the statement:

> Cinder concrete at high temperature transmits heat rapidly and loses strength.*

This may be compared with some statements of other experienced observers. For example, Prof. Ira H. Woolson (elsewhere quoted by Committee E-4) said at the building code hearing in New York City, June 1909:

> The best fire-resisting material used in construction today is cinder concrete without any qualification.

The divergency of opinion is naturally explained by the fact that the material is not clearly defined. We speak of the fire resistance of concrete—yet no two concretes behave alike. Cinders as an aggregate are less clearly defined than others—it is natural that the resulting concrete will not be uniform to the second decimal.

When cinders are improperly used (poorly graded aggregate, careless mixing and making) the resulting concrete may be so open-textured that it permits free passage of air. The heat transfer through such concrete is naturally rapid, and it is quite possible that exposed carbon particles in the interior may burn.

As an example of such poor cinder concrete may perhaps be cited the three specimens of Technologic Paper No. 130 of the

*It is unfortunate that some qualification and elucidation of this statement by Committee E-4, did not have equal prominence with the original report. It is in statements by N. D. Mitchell, A. C. I. *News Letter*, Nov. 1929, p. 11.—Editor

Bureau of Standards (referred to by Committee E-4). The cinder concretes (sand-cinder) of that paper are described as follows:

> Cinder concretes Nos. 1291 and 1292 were extremely weak and friable. The appearance of the concrete indicated too great a proportion of fine material.
>
> Cinder concrete No. 1291a was an extremely rough specimen because of a deficiency of fine material.

Such mixtures are not typical of the cinder and sand-cinder concrete of today and were evidently prepared by someone who was not familiar with designs involving this type of aggregate.

In well designed cinder concretes the fire resistance has been quite remarkable. Tests have been made at the Underwriters' Laboratories by the cinder unit industry and have shown a resistance to fire and water, and a retardance of heat transfer greater than in similar sand concrete units. As previously mentioned, the presence of coal and coke does not materially lower the fire-resistance, within limits. It lowers the strength of the concrete after fire exposure, but even a unit of pure coal and coke aggregate was not destroyed in a test reaching its temperature endpoint (8-in. hollow wall) in 3 hours and 49½ minutes.

In sand-cinder concrete, the evidence of the test house of Columbia University is of particular interest. The permanent walls of this house are built of 1:2:5 sand-cinder concrete. Professor Woolson cited this house at the 1909 hearing on the New York building code:

> For these test buildings we have tried brick many times. Brick lined with cinder concrete 3 or 4 inches thick and cinder concrete buildings we have also tried, and as a result of having built several of these buildings I have arrived at the conclusion that the only and best material to use for a building of that character is the cinder concrete.

Before the convention of the National Association of Cement Users, 1911 (Vol. 11), Professor Woolson stated of this test structure, which had then been subjected, in the aggregate, to 24 hours of firing at an average temperature of 1700 degrees, with water applied at the end of tests:

> Today (Dec. 13, 1910) the building is standing in as perfect condition as it was the day it was built, having some few temperature cracks which, of course, are due to that very excessively severe treatment.

Concerning this building, Rudolph P. Miller stated in 1925 (in report to Crozier-Straub, Inc.):

Since that statement was made the same structure has passed through over forty hours of such severe treatment in good condition, without the need of repairs.

Perrine and Strehan open the discussion of fire resistance of sand-cinder concrete by saying:

To any one at all familiar with the effect on cinder concrete of flames at high temperature followed by the application of cold water at high pressure, little other evidence is necessary to prove its efficiency under this treatment.

Inasmuch as Committee E-4 is chiefly concerned with the heat transfer through the material, it may be well to point to the tests by Humphrey, as reported in Bulletin 370 of the U. S. Geological Survey. If the writer reads the graphs correctly, 8-in. solid concrete blocks in similar fire tests showed about the following:

	Unexposed Face of Wall	
	Initial Temperature	After 2 Hours
8-in. granite concrete	18 deg. C.	58 deg. C.
8-in. gravel concrete	23 deg. C.	56 deg. C.
8-in. sand cinder concrete. 1:2:4, with 24.5 per cent unconsumed carbon	28 deg. C.	43 deg. C.

Weighing all the evidence at hand, both cinder and sand-cinder concrete appear to possess a remarkable degree of fire-resistance—provided they are well made. 8-in. hollow cinder blocks, 1¾-in. web, not more than 35 per cent combustible, 1 cement to 7 dry rodded cinders, are given a 3-hour rating by the Underwriters' Laboratories, and are certified as such. Cinder concrete building units in fire tests, and in actual fire exposures, have shown a remarkable absence of cracking and spalling, both when exposed to fire and to water. The status of sand-cinder concrete is not as clearly defined—but the fire record of this material has been truly remarkable.

Heat Transmission

The 1930 Guide of the American Society of Heating and Ventilating Engineers lists the following heat transmission coefficients for "cinder concrete" (without specifying the composition of the material):

Solid cinder concrete. Conductivity "k" (per 1 in. thickness) 3.33

12-in. hollow cinder block, four cores, three longitudinal webs. Conductance "C" (face to face) 0.28

8-in. hollow cinder block, two cores. Conductance "C" (face to face) . . . 0.45

These values check well with those obtained in laboratory tests witnessed by the writer. The tests are summarized as follows:

At the University of Toronto, Prof. R. W. Angus[17a] conducted tests for Canadian Cincrete, Ltd. The cinder units were built into a wall, 7 blocks high and 3 blocks wide, having a total area of 20.31 sq. ft. The wall was set up in a large box in such a manner as to divide the box into two compartments. One of these compartments communicated directly with the cold room of a refrigerating machine, while the other compartment was heated electrically. Fans distributed the air over the surface of the wall.

The air on the warm side was maintained at approximately 70 deg. F. while the air on the cold side was maintained at about 20 deg. F. When the temperatures on each side remained constant, all the heat put into the warm compartment was transferred through the wall. From this the conductance and transmittance may be calculated, when knowing the temperatures of the air and the surfaces of the wall.

The units under test were 8 x 8 x 16-in. hollow cinder block, having three oval cores and approximately 40 per cent air space. The design of mix is not definitely known, but the units were made from railroad cinders and designed to meet the code requirement of 1,000 lbs. per sq. in. This should represent a relatively dense cinder concrete.

8-in. hollow cinder block wall exposed

Conductance "C" (surface to surface)	0.423
	0.415
Transmittance "U" (air to air)	0.340
	0.335

At Penn State College, Prof. F. G. Hechler[17a] conducted tests for National Building Units Corp. Two identical walls were built of the cinder units under test, each 7 units high and 3 units wide. A hot plate was fitted tightly between the two walls which were clamped together. The hot plate was surrounded by a guard ring which was maintained independently at the same temperature as the plate, so that all heat flowing from the plate passed through the two walls. The hot plate was electrically heated and the heat input as well as the temperature was known. Knowing the temperature of the outside surfaces of the walls, the conductance "C" may be computed.

8 x 8 x 16-in. hollow cinder units were tested. The units were made from railroad cinders, mixed 1:7, to pass a strength requirement of 700 lbs. per sq. in. They had two cores, or about 33 per cent air space.

8-in. hollow cinder block wall exposed

Conductance "C" (surface to surface)	0.457

This value was determined at a mean temperature of 62.7 deg. F. As the temperature was increased, the conductance increased. At 79.8 deg. F. it became .489. This must be caused by increased radiation and convection through the hollow space.

It was also found that the conductance was less when the flue formed by the air space of the hollow units was sealed at the top than when it remained open. The convection of the hollow space evidently increased when the flue acted as a chimney.

4 x 8 x 16-in. solid cinder units, representing the same type of cinder concrete, were also tested, laid in walls of the same size.

4-in. solid cinder concrete exposed
Conductance "C" (surface to surface)
Mean temperature 56.3 deg. F.....1.005
Mean temperature 68.8 deg. F.....1.04
Mean temperature 85.3 deg. F.....1.06

The Conductivity "k" per 1 in. thickness of this solid wall may be figured as 4 x 1.00 = 4.00 (4.25 for the highest temperature). This value is higher than that of 3.33 used in A. S. H. V. E. Guide—but it must be remembered that the 4-in. wall contained mortar joints, comprising probably 10 to 15 per cent of the wall area. If we accept the value of 8.3 (generally used) for the conductivity of this mortar, the value of 3.33 for the conductivity of the cinder concrete checks well with the test.

It may not be amiss to point out that considerable confusion prevails in many discussions of heat transmission data. The nomenclature is not uniform, and it is not always made clear to what conditions the coefficients reported belong. It is quite common, for example, to apply data obtained on one small unit to larger wall assemblies of that unit, or to compute the coefficient for a hollow wall on the basis of data for the solid material without taking into account the air space. In the present paper, the writer is adopting the nomenclature of the A. S. H. V. E.

As far as sand-cinder concrete is concerned, Prof. C. L. Norton reports on a 1:2:4 mixture, tested by the hotplate method. Without giving details regarding the make-up of the concrete, Norton states the conductivity as follows:

1:2:4 sand cinder concrete
Conductivity "k" (per 1 in. thickness)
Mean temperature 122 Deg. F.....2.35

On the other hand, the Guide of the A. S. H. V. E. quotes Peebles as follows:

(Sand-) cinder concrete (110 lbs. per cu. ft.).
Conductivity "k" (per 1 in. thickness)
Mean temperature 75 deg. F.....5.20

These two values are far apart. Aside from probable differences in method (the one used by Peebles is not stated), it is possible that the concretes may have been quite different in their make-up. Norton reports the conductivity for 1:2:5 stone concrete to be 6.27, while Peebles reports the value for stone concrete weighing 145 lbs. per cu. ft. as 6.30. Willard, Lichty and Harding report the value for 1:2:4 stone concrete as 8.30 (140 lbs. per cu. ft., mean temperature 110 deg. F.), and this is the value used in the calculations of the A. S. H. V. E. Guide. It would appear that the value reported by Norton is too low for the average product—perhaps the value reported by Peebles is too high. The method employed and the nature of the material must be known in detail if one is to judge the significance of heat transmission coefficients.

The heat flow through a wall depends to a great extent upon the pore structure of the wall material. If the pores are large and open, free passage of air may occur, and the heat transfer is considerable. The writer knows of cinder concrete, molded on a primitive hand press, which was so porous that a 30-mile wind could be felt through an 8-in. wall of the material. This, however, is not the type of concrete here considered. Following the recommendations for grading, mixing and making, here proposed, the resulting concrete will have the porous, cellular particles of cinders embedded in a relatively dense matrix; the cells will contain "dead air," and contribute to the insulative quality of the material.

The addition of sand will affect the pore structure. There are no data available to show to what extent. Such data can only be produced by carefully designing the concrete mixtures and keeping all variables of test under control.

The pore structure of the concrete is of particular importance when fire retardance is concerned. An open-textured cinder concrete will not only permit a rapid transfer of heat, but may also permit the fire to affect the interior of the concrete. When discussing fire resistance, it was pointed out that such a concrete might well justify the opinion which certain authors hold concerning cinder concrete, but such concrete is not common.

Sound Absorption

The problem of reducing sound reverberation in large rooms, and sound transmission through partition walls is involved. Some experimental data are available regarding cinder concrete building units, but these data will not be reviewed here. Reference is made to manufacturer's publication.

The writer has observed, on numerous occasions, the marked sound absorption of cinder concrete, either exposed or painted. In one building particularly where some of the cinder unit walls were painted while others were plastered, the difference in sound reverberation between rooms of different wall finishes was immediately noticeable. The texture of the cinder concrete—open, when compared to that of ordinary plaster—should be of value in improving the acoustics of the room.

The transfer of sound through a material is naturally considerable when the pore structure is such as to permit free passage of air. Plastered partitions of cinder concrete appear to have given a high degree of satisfaction in many jobs like hotels, hospitals or apartment buildings where sound insulation is an important matter. The writer bases this statement on personal observations, rather than on quantitative tests.

There are apparently no data available regarding the behavior of sand-cinder concrete in this respect.

The Material Generally

Cinder concrete, properly made, will receive and hold nails to an extent depending upon the nature of the cinders, the design of mix, the method of making and the age of the concrete. The advantage of nailable concrete is particularly obvious in walls where the direct fastening of furring strips, plastering grounds, and the like, results in marked economies of construction.

Nailability depends upon the nature of the aggregate, cellular anthracite cinders here being preferable to denser, well clinkered bituminous cinders. Such other factors as cement/aggregate proportion, amount of mixing water, degree of compacting of concrete, naturally affect nailability as they affect the pore structure of the material. The fact remains, however, that cinder concrete building units as usually manufactured (proportions 1:7 to 1:10) to meet a 700 lbs. requirement, are readily nailable

at the time when they are usually placed in the wall. Numerous tests have been made on the holding power of such nails, but the tests have not been devised to show the relation of nailability to the other characteristics of cinder concrete. Addition of sand to the aggregate naturally reduces nailability; quantitative data are not available.

Machine-made cinder concrete building units, made from a relatively dry, non-plastic mix, have an interesting surface texture, caused by the roughness of the particles of aggregate and developed through the method of moulding. Unusual architectural effects have been obtained by painting this surface without destroying the texture. The adhesion of plaster, forced into the crevices of the surface, appears to be considerable; demonstrations of the strength of mortar bond have been made, but no accurate test data are available. The surface texture of wet cast, denser sand-cinder concrete is less interesting.

History and Bibliography

It is not within the scope of this paper to deal with the history of the use of cinders in concrete, beyond the incidental mention of certain facts already considered. It is only just, however, to point out two developments which have greatly influenced the status of the material in the United States.

First, as to sand-cinder concrete. The recognition given to this material in the City of New York, through the work of Ira H. Woolson, Rudolph P. Miller, A. L. A. Himmelwright, Harold Perrine and George E. Strehan, is an important factor in its development, often in the face of the strongest opposition. Fifteen years ago, sand-cinder concrete was a very live subject of discussions—for some unknown reason it lost its actuality.

Second, as to cinder concrete building units. The work of F. J. Straub in putting his "cinder block patent" into practical use is to a great extent responsible for the development of a cinder unit industry. It has been the subject of much litigation—and the writer has been careful to avoid, in this report, all controversial issues. These are reserved for other forums than that of the A. C. I. But the writer wishes to acknowledge his debt to F. J. Straub, and his associates in the present industry, who had the vision and the common sense necessary to overcome a multitude of prejudices.

In the following bibliography, the writer has listed only the papers which he considers to be of primarly interest to the student of the subject. So much of the published data are merely repetitions, and a complete bibliography, including every publication in which cinders are mentioned, would fill many pages and would be of little value.

1. Charles L. Norton—*Engineering News*, October 1902; *Engineering News*, January 1904. Tests to determine the corrosion of steel in concrete.

2. R. T. Surtees—"A few tests and experiments with reinforced concrete." *Engineering Record*, December 14, 1907. Tests to determine the corrosion of steel.

3. Richard L. Humphrey—"The strength of reinforced concrete beams." U. S. Geological Survey, Bulletin 344, 1908.

4. R. L. Humphrey and L. H. Losse—"The strength of reinforced concrete beams." U. S. Bureau of Standards, Technologic paper No. 2, 1912. Two papers reporting interesting data on the design of sand-cinder concrete mixtures.

5. D. B. Butler—"The dangers of breeze concrete." The Journal of the Society of Architects, April 22, 1909. Expansional movement.

6. Ira H. Woolson. *Cement Age*, October 1909. Testimony before the New York Building Code Commission, summarizing experience with sand-cinder concrete in fire tests. Numerous reports by Woolson on fire tests involving sand-cinder concrete will be found in various technical journals, but are not listed here.

7. Charles L. Norton—"Some thermal properties of Concrete." Proceedings, National Association of Cement Users, 1911, p. 78. Heat transmission of sand-cinder concrete. Fire resistance discussed.

8. Ira H. Woolson and Rudolph P. Miller—"Fire tests of floors in the United States." Proceedings, International Association for Testing Materials, Paper XXVII, 2nd. section, 1912. Report on a large number of fire tests involving sand-cinder concrete floors of different designs.

9. Guy B. Waite—"Cinder Concrete Floors." Transactions, American Society of Civil Engineers, Vol. LXXVII, p. 1773, 1914. Tests on reinforced sand-cinder concrete floors.

9A. M. C. Tuttle—"Bibliography of Corrosion of Iron and Steel in Cinder Concrete." *Engineering News*, April 18, 1912.

10. Harold Perrine and George E. Strehan—"Cinder concrete floor construction between steel beams." Transactions, American Society of Civil Engineers, Vol. LXXIX, p. 523, 1915. Tests on reinforced sand-cinder concrete floors, presenting a review of the state of the art, and followed by interesting discussion.

11. H. Nitzsche—"Untersuchungen an Kesselschlacke für Betonzwecke." *Zement*, March 1920. A comprehensive paper on cinder concrete.

12. Arthur Holmes—"Geological and Physical Characters of Concrete Aggregates." British Fire Prevention Committee, Red Book No. 256, 1921. Data on various types of cinders.

13. H. Nitzsche—"Die Bedutung der Schwefelsauren Salze der Kohlenschlacken für die Beständigkeit der Schlackebeton-Erzeugnisse und für deren Verhalt bei wechselnder Feuchtigkeit." *Zement*, 1925, p. 338. Expansional movement due to the presence of sulphur compounds. Also tests on washing of cinders.

14. F. M. Lea and F. L. Brady—"Slag, Coke Breeze and Clinker as Aggregates." Department of Scientific and Industrial Research, Building Research, Special Report No. 10, London 1927. Brief review of these aggregates.

15. F. M. Lea—"Investigations on Breeze and Clinker Aggregates." Department of Scientific and Industrial Research, Building Research, Technical Paper No. 7, London 1929. A very comprehensive study of the effect of coal and sulphur compounds upon the permanence of cinder concrete.

16. F. L. Brady—"The Corrosion of Steel by Breeze and Clinker Concretes." Department of Scientific and Industrial Research, Building Research, Special Report No. 15, London 1930. Test on corrosion of steel in contact with cinders and cinder concrete.

17. National Building Units Corporation, Philadelphia, has published considerable data relating to cinder concrete building units manufactured under the Straub patent.

a. Technical Series: Bulletin No. 1: "Sound Absorption of Cinder Concrete Building Units."

b. Technical Series: Bulletin No. 2: "Three hour fire and water test made upon walls of 8 inch and 4 inch hollow Cinder Concrete Building Units."

c. Technical Series: Bulletin No. 3: "Thermal insulation of masonry walls."

d. Report of Underwriters' Laboratories on Straub Hollow Cinder Concrete Building Blocks, Retardant No. 1429, 1922. With supplementary data revising specifications, May 14, 1930.

e. Memorandum on Cinder Concrete Building Units and their place in modern building codes.

Many data regarding cinder and sand-cinder concrete, including a very complete list of patents, will be found in the Transcript of Record in the case of Crozier-Straub et al. vs. Graham et al., United States Circuit Court of Appeals, Third Circuit, March Term 1928.

Admixtures And Workability of Concrete

BY G. M. WILLIAMS*

*Two critics who considered this paper on behalf of the Publications committee recommended that to increase its practical value the author should designate the admixtures by name, rather than disguise them with numbers. Professor Williams has graciously responded with the names and they appear in a footnote on the page with Table 1, in which the admixtures are first listed by number. In justice to the author and to producers of admixtures here given the lower ratings, the author's viewpoint, with respect to his contribution, requires emphasis. Professor Williams' brief paper is to be read as a suggestion for a different approach to the study of admixtures and not for the limited data it presents on the relative merits of the admixtures he used in his studies of a method of test. The author writes: "My purpose was to outline a method of test for the rating of admixtures rather than to discuss the merits or demerits of any particular one or group * * * A satisfactory test should be established first * * * Had I intended to compare the admixtures in this manner I would have included several others which have been used commercially, one of them approaching fairly near to No. 6 in efficiency * * * I included a minimum of test data in tabular form to make comparisons easier * * * Strength, porosity and permeability must be taken into account in any final rating of an admixture, but I felt that to discuss these factors at length would only divert attention from the test methods proposed, toward the admixtures themselves. The statement that admixtures tested 'do not appreciably affect strength or permeability when used in the amount or proportions specified by the manufacturer' briefly sums up any discussion which I might make in connection with these factors." Thus while we*

*Professor of Civil Engineering, University of Saskatchewan.

may look forward to further data on admixtures from Professor Williams, his present contribution is aimed at the method of studying their values and not at the relative values obtained from such study.—EDITOR

IN RECENT YEARS the use of admixtures in concrete has greatly increased due to advantages which their use has demonstrated under certain conditions of concrete practice. The advantages advanced for admixtures, by the manufacturers, vary, but in general, improvement in strength, water-tightness and workability is claimed. The effect of admixtures on the first two qualities can be measured by testing procedure already standardized or developed but their effect on workability is not so easily measured owing to different views as to what is meant by workability as well as absence of well established test to measure this property.

By workability, the writer means the ability of a mixture to remain homogeneous and free from segregation during the process of transportation and placement. Workability is a relative condition. A concrete might have satisfactory workability under certain conditions but be unworkable or segregate badly under others. While this definition may not agree with various laboratory definitions of the term workability, it is believed that the statement defines workability from the view point of the practical concrete maker. In practice a segregating or poor working mixture may result, after transportation difficulties have been overcome, in stone pockets, laitance, porosity, excessive permeability and low compressive strength as well as increase the cost of handling and placing if properly attempted. A homogeneous, workable mix will after placement be free from the foregoing visible and hidden defects. The practical worker will refer to such a mix as "workable."

Non-workability or segregation may be due to low mortar content, an excessively wet or soupy mortar, an excess of coarse aggregate or to a combination of these characteristics. Low mortar content may be due to low cement content (lean mix) with an aggregate which may be well balanced with respect to relative quantities of fine and coarse aggregate. The use of

additional mixing water to obtain flowability as well as workability results, in turn, in an excessively wet segregating mortar which flows away from the coarse aggregate or which permits the coarse aggregate to settle out towards the bottom of the container. Fairly rich mixtures of low sand-gravel ratio may also lack a sufficient volume of mortar to carry the volume of coarse aggregate, and the attempt to attain sufficient flowability to give easy placement results in segregation. The flowability of a mass of concrete is due entirely to the mortar content and when this is insufficient in quantity or too wet to adhere properly to the coarse aggregate particles, segregation will result.

A non-segregating homogeneous mix must have a sufficient volume of mortar to float and hold in suspension the coarser aggregate particles. This implies not only a relatively large mortar volume but a sticky plastic mortar which will adhere to and carry along the aggregate particles without segregation when the mass is set in motion.

For concrete made with given materials workability may be improved by increasing the volume of mortar which in turn will permit the use of a less fluid, more adhesive mortar without reducing the flowability of the whole. The addition of any material to the mix which will increase the relative volume of the mortar will therefore improve workability and that material which for a given weight will produce the greatest mortar volume will probably be most efficient providing the other properties of strength and watertightness are not injured.

The writer has tested a number of admixtures, including portland cement itself as an admixture, and believes that the following conclusions with respect to their effect on workability are justified.

1. All admixtures tested, including portland cement as an admixture, increased relative mortar volume, tended to reduce segregation, and showed merit as a means of increasing workability although not to the same degree.

2. The relative efficiency of admixtures in increasing workability (reducing or preventing segregation) is measured by the relative volumes of paste produced when equal weights to admixtures are made into pastes of approximately the same flowabilities.

Six admixtures of Canadian and American manufacture, in-

cluding portland cement were tested. They were all of the practically insoluble, finely divided powder form, and ranged in composition from silica and lime to clay in the form of bentonite. When prepared in neat paste form with water they varied considerably in adhesiveness or stickiness, especially with low water contents, but much of the stickiness was lost when sufficient water was added to bring them to the approximate consistency which they would have as a part of the mortar volume of a workable concrete mixture.

The following results will indicate the test methods employed to furnish the conclusions outlined above.

TABLE 1—RELATIVE VOLUMES OF PASTES OF SAME FLOWABILITY PRODUCED WITH EQUAL WEIGHT OF ADMIXTURES

Admixture*	Relative Amount of Mixing Water Required	Relative Volume of Paste Per 100 Grams of Admixture
1 (cement)	1.0	1.00
2	3.2	1.95
3	3.3	1.95
4	2.4	1.62
5	3.4	2.00
6	13.0	5.70

In Table 1 it is seen that all admixtures were more efficient than cement as measured by volume of paste produced. Four of the admixtures are grouped fairly closely while No. 6 falls in a third group.

Equal weight of admixtures were then added to identical cement-sand mortars and the volume increase due to the presence of the admixture was determined as listed in Table 2.

TABLE 2—RELATIVE VOLUME INCREASE PRODUCED IN IDENTICAL CEMENT-SAND MORTARS TO WHICH EQUAL WEIGHTS OF ADMIXTURES WERE ADDED. ALL MORTARS WERE OF SAME FLOWABILITY AFTER ADMIXTURES WERE ADDED

Admixture*	Relative Volume of Mortar	Relative Increase in Mortar Volume Per 100 Grams Admixture
None (Plain Mortar)	1.000	—
1 (Cement)	1.040	1.0
2	1.045	1.1
3	1.045	1.1
4	1.052	1.3
5	1.060	1.5
6	1.165	4.7

*No. 1 = Canada Portland Cement; No. 2 = Vankelad A; No. 3 = Vankelad B; No. 4 = Barnsdall Admixture; No. 5 = Hydrated Lime; No. 6 = Celite. See Editor's note at beginning of this paper.—EDITOR

Comparing the results in the right hand column of Table 2 with the same column of Table 1 it is apparent that the relative volume increases in these particular mortars were not of the same magnitude as were those found for the relative volumes of neat pastes but the values are somewhat proportional. Apparently some of the paste has gone to fill void space and the rest swells the volume of the mortar.

Tests were then made to measure the actual segregation which occurred in identical concrete mixtures to which were added equal weights of the different admixtures. Flowability, as measured by spread of mortar on the flow table was made constant by the addition of sufficient water in each case. The mortar content of the batch was not excessively wet, but rather sticky and plastic, and segregation was obtained by the use of an excess of $\frac{3}{4}$ in. to $\frac{3}{8}$ in. aggregate. Amount of segregation was measured by weighing the coarse aggregate particles which rolled from the mass beyond the mortar line on the flow table. It was difficult to obtain concordant results in repeated tests of any one batch or in tests of successive batches since it was difficult to select and properly place a fair sample of the batch in the flow table mold, all of the batches segregating to varying degrees. The results shown in Table 3 do indicate much and varying segregation, but do not permit of rating the admixtures, which in the other tests are shown to be of somewhat equal merit.

TABLE 3—PERCENTAGE OF COARSE AGGREGATE SEGREGATED FROM CONCRETE MASS AND PERCENTAGE OF MORTAR IN CONCRETE

Admixture*	Ratio Volume Mortar to Volume of Concrete	Percentage (by weight) of Coarse Aggregate Segregated from Mass
None (Plain Concrete)	.472	13.13
1 (Cement)	.479	11.30
2	.482	9.91
3	.482	10.57
4	.482	9.90
5	.487	9.92
6	.514	4.95

Since it is difficult to handle and sub-divide a segregating concrete the values above in Table 3 should be considered only approximate, but they indicate greatest segregation for the plain concrete, slightly less for cement as an admixture, still less of the group of four admixtures and least for admixture No. 6.

To determine definitely whether or not the value of an admixture in promoting workability is proportional to its relative paste volume concretes similar to those included in Table 3 were prepared, but instead of using equal weights of admixtures the amounts were increased so as to make the paste volumes of admixtures equal. From the results of Table 1 the relative weights of admixtures required were found to be as follows:

Admixture*	Relative Weights of Admixtures to Furnish Equal Volumes of Admixture Paste
1 (Cement)	5.7
2	3.0
3	3.0
4	3.5
5	2.8
6	1.0

TABLE 4—PERCENTAGE OF COARSE AGGREGATE SEGREGATED FROM CONCRETE MASS AND PERCENTAGE OF MORTAR IN CONCRETES

Admixtures Added to Furnish Equal Volumes of Admixture Paste

Admixture*	Relative Weight of Admixture Used	Ratio Volume Mortar to Volume of Concrete	Percentage by Weight of Coarse Aggregate Segregating from Concrete
1 (Cement)	5.7	.517	4.20
2	3.0	.510	4.85
3	3.0	.514	4.95
4	3.5	.514	4.73
5	2.8	.518	5.00
6	1.0	.517	3.98

Results of tests in Table 4 indicate that the additions of admixtures in quantities to produce equal volumes of admixture pastes do furnish concretes of practically equal mortar contents and equal workability as measured by segregation of coarse aggregate.

From the foregoing test results it would appear that the mortar content of a batch, for these particular aggregates, should be well in excess of 50 per cent to insure workability, and for concretes which may be subjected to considerable jarring and shaking during transportation not only a relatively large volume of mortar but in addition a mortar of comparatively low flowability would be required.

While admixtures are relatively more efficient than cement in obtaining a concrete of workability suited to any given condition

of transportation and placement the writer does not suggest that their use is always necessary or advisable. For concrete of any required cement content first attention should be directed to the most desirable sand-gravel ratio. Relative increase in sand content increases mortar volume which in turn permits the use of a less fluid mortar, thereby increasing workability and postponing likelihood of segregation. After adjusting the sand-gravel ratio and finding segregation under the conditions of use, or that the mix is near the segregation limit the use of an admixture will be advantageous. In work where a surface as true as the face of the mold is desired or in any case where the available fine aggregate is coarse and harsh, lacking in fine particles, the use of an admixture may be of value in obtaining satisfactory results.

The factors of strength and watertightness should not be forgotten. Generally these properties cannot be sacrificed to obtain workability. Tests which the writer has made quite definitely indicate that admixtures do not appreciably affect strength or permeability when used in the amounts or proportions specified by the manufacturer. In cases where excess quantities of an admixture are needed to produce satisfactory workability the possibility of reduction in strength and watertightness must receive consideration. On the basis of workability alone, the data in Table 1 would indicate the use of an admixture to be dependent mainly upon the relative cost of cements and admixtures. The use of admixtures 2 to 5 would not be economical unless their cost were less than twice, and No. 6 less than five or six times, the cost of portland cement. Consideration should also be given to the fact that cement not only increases workability as an admixture but at the same time it definitely increases strength and impermeability.

Readers are referred to the JOURNAL *for May 1931, for discussion which may develop. Such discussion should reach the Secretary by April 1, 1931. (Normally the discussion of a paper in the February* JOURNAL *would appear in June but is in this instance scheduled for May to combine it with discussions of "Slump Tests and Flow-Table Determinations"—two papers appearing in the January* JOURNAL, *involving in part a different approach to the same fundamental problem.)*

A Comparison of Continuous With Batch Mixers In Plant Operation*

BY BENJAMIN WILK†

INTRODUCTION

The advantages of a batch mixer as against a continuous mixer have been the subject of considerable discussion among concrete products manufacturers for many years. The batch mixer has generally been favored because of its longer mixing time and the simplicity with which aggregates can be proportioned into the mixer. The continuous mixer has had its advocates because of the greater amount of concrete that could be mixed per day per dollar of investment.

Until recent years continuous mixers were built generally with short mixing troughs, many of them not over three feet long and with uncertain feeding devices. Mixing time was therefore very short, frequently less than one-half minute. This type of continuous mixer has not been very widely used. Improvements in design, however, during recent years. have developed continuous mixers with more accurate feeding devices and longer mixing troughs.

To determine the comparative value of the improved continuous mixers as against batch mixers a series of tests was made in the plant of the Standard Building Products Co., Detroit, Mich., in 1929 and 1930. Batch mixers had been used in this plant for several years, but a machine requiring a continuous, large supply of concrete was being installed and the question arose whether to use a continuous mixer of a moderate size, or a very large batch mixer. To obtain a practical comparison a continuous mixer was installed. Tests were carried on under the

*For presentation and discussion at the 27th Annual Convention.

†General Manager, Standard Building Products Co., Detroit.

supervision of the writer, assisted by J. Paul Wilson, engineer of the Concrete Products Association of Detroit.

TEST PROCEDURE

Comparative tests were made over a period of six months at intervals of approximately 30 days commencing in November, 1929, and ending in May, 1930. Each test consisted of two successive batches made in the batch mixer followed by an equivalent amount in the continuous mixer. Control was obtained by measuring aggregates in a one cubic foot box for the batch mixer and by determining the proper setting of feeding devices on the continuous mixer based on the yield in terms of block per sack of cement for the previous day's run.

Each batch consisted of three wheelbarrows of aggregate and one sack of cement. Each wheelbarrow contained exactly three cubic feet of aggregate. Batches were mixed three minutes dry and three minutes wet. This mixing procedure was used for the batch mixer because it was found to have given practically the best results in a series of tests made by the Committee on Plant Operation of the American Concrete Institute, reported in the 1929 Proceedings. The amount of water was just enough to give a water-web mark on the finished product. The mixed concrete dropped directly into a hopper feeding the block machine. The yield from these two batches was 39 block, or an average of 19½-eight in. block per sack of cement.

Aggregate and cement were fed simultaneously through the continuous mixer. Two sacks of cement were put into the cement hopper and a measured flow of aggregate started into the aggregate hopper. The mixer was allowed to run until all of the cement from the two sacks ran out. The aggregate and cement was mixed dry in the mixing trough for a distance of approximately thirty-two inches, or one-third the length of the trough. At this point water was added in just sufficient quantity to give a water-web mark on the finished product. Mixing time in the continuous mixer was determined by dropping color in with the cement and also by small colored pieces of wood. Mixing time was slightly less than one minute. As the mixed concrete dropped off the end of the trough it was shoveled into wheelbarrows and immediately moved to the hopper above the block machine. The

yield from the continuous mixer varied from 38 to 43 eight in. block per two sacks of cement, or an average of 20¼ block per sack. The machine in which the block were made was directly below the batch mixer. The block from both mixers were made by an experienced operator in the usual plant practice. They were placed on two racks set in the curing room, one behind the other, so that all the block had the same curing conditions.

AGGREGATE

The aggregate used was washed sand and pebbles so combined as to produce a fineness modulus of 3.4. Sieve analyses of the combined aggregate made at intervals, checked very closely with the calculated finess modulus.

DESCRIPTION OF MIXERS

The continuous mixer was an Eberling driven by a 5 h. p. motor. The cement hopper had a capacity of slightly more than two sacks of cement, and the divided hopper for the aggregate held slightly more than four cubic feet. A chute ran directly from an overhead bin to the aggregate hopper, so that there was a continuous flow of aggregate into the hopper. The feeding belts under the hoppers were 18 in. wide. The roller under the cement hopper was shaped as an octagon. This caused a slight movement of the belt as it passed under the cement hopper and thereby eliminated any arching that might have developed. The mixing trough was semi-circular with a diameter of 20 in. and an overall length including dry and wet mixing of 9 ft. There were 34 double blades, each one approximately 17 in. long, revolving at a speed of 35 r. p. m., mounted on a 3-in. shaft. Theoretically each particle of concrete was pushed ahead and therefore mixed 68 times in traveling through the mixing trough. The volume of concrete mixed per minute was approximately 4 cu. ft.

The batch mixer was a standard oversize 9 cu. ft. Blystone side dumping mixer equipped with six blades. A 15-gal. water tank directly above the mixer was used to control the amount of water added per batch.

CURING AND TESTING OF BLOCK

The block were cured for twenty-four hours in steam at a temperature of approximately 90 deg., and then cured in air

until 72 hours before time of testing. They were then delivered to the laboratory of the Concrete Products Association of Detroit, and stored inside at room temperature for the remaining 72 hours before they were broken. Test results are shown in the following table. "C" results are from the continuous mixer, "B" results from the batch mixer.

SEVEN DAY TESTS

Series No.	C-1	B-1	C-2	B-2	C-3	B-3	C-4	B-4	C-5	B-5
Yield Per 2 Sacks	38	40	42	39	40	39	43	39	41	39
	770	690	680	740	560	580	740	820	560	660
	830	730	640	770	530	580	710	790	610	640
	820	710	610	700	550	600	690	790	630	580
	807	710	643	736	547	587	713	800	600	627

FOURTEEN DAY TESTS

Series No.	C-1	B-1	C-2	B-2	C-3	B-3	C-4	B-4	C-5	B-5
Yield Per 2 Sacks	38	40	42	39	40	39	43	39	41	39
	980	1060	840	1010	780	750	510	910	750	600
	1060	770	840	1000	810	790	530	930	630	620
	1020	915	840	1005	795	770	520	920	690	610

The following table shows comparisons after results were computed on the basis of 39 block per two sacks.

SEVEN DAY TESTS			FOURTEEN DAY TESTS		
Series No.	Yield 2 Sacks	Av. of 3 Block	Series No.	Yield 2 Sacks	Av. of 2 Block
C-1	39	780	C-1	39	990
B-1	39	740	B-1	39	940
C-2	39	690	C-2	39	910
B-2	39	740	B-2	39	1000
C-3	39	560	C-3	39	810
B-3	39	590	B-3	39	770
C-4	39	790	C-4	39	570
B-4	39	800	B-4	39	920
C-5	39	630	C-5	39	730
B-5	39	630	B-5	39	610

The grand average for the seven day block from the continuous mixer is 690 lb., from the batch mixer 700 lbs. The grand average for the fourteen day block from the continuous mixer (including

the unusually low figure of 574 lb. in Series 4) is 802 lbs., and from the batch mixer 848 lbs.

The final testing was at fourteen days because in the Detroit market it is desirable to be able to deliver block on the job at the end of fourteen days which have a strength above the 700 lbs. requirement.

CONCLUSION

The test results indicate that the variation between the strength of the block made from the continuous mixer of the long trough type and the usual batch mixer is very small, although the time of mixing in the continuous mixer is approximately one minute as against six minutes mixing in the batch mixer. Evidently the concrete in this type of continuous mixer is given a thorough mixing by the large number of paddles and the large number of revolutions per minute.

Readers are referred to the JOURNAL *for June 1931 for discussion which may develop. Such discussion should reach the Secretary by May 1, 1931.*

Discussion of Report of Committee 201

"Blast Furnace Slag as Concrete Aggregate"*

BY L. W. WALTER†

The completeness of this report, dealing as it does with the processing of slag, its physical and chemical properties and with the properties of slag concrete, entitles it to a rating as probably the best single contribution to concrete literature ever presented on this subject.

This may well be true, as the time was opportune, and the National Slag Association had made available to interested committees engaged in formulating specifications for concrete and concrete aggregates a vast amount of information covering test data and service history of slag and slag concrete. This information, furnished through symposiums issued at regular intervals over the last two years, covers more than 500 pages and represents, as far as I can recall, the most comprehensive collection of data on any single concrete aggregate.

This brings to my mind the very meager published information obtainable on slag and slag concrete when, early in 1914, I attempted a collection of available data preliminary to a survey of concrete structures in which slag had been used as a coarse aggregate. At that time, if you wanted real information about slag and needed it, you had to go into the field and get it. I had previously known of its use by iron and steel companies to the exclusion of other aggregates. I had interviewed their engineers, had observed their methods, and examined the materials used. These included natural, puzzolan, and portland cements, granulated slag as a substitute for sand, and the use of coarse aggregate ranging from bank-run of air-cooled slag to pancake slag.

*Journal A. C. I., October, 1930; *Proceedings*, Vol. 27, p. 183.
†Inspecting Engineer, Erie Railroad Co., Jersey City, N. J.

The pancake slag was sufficiently cooled with a water spray to produce brittleness which would encourage its breaking up when dropped into steel gondola cars. The cars, after loading, were shifted to the concrete mixer and the slag used in cold weather before the initial heat had passed off—in some cases from the blast furnace into the concrete on the same working day.

My survey in 1914 of slag concrete structures was largely but not entirely confined to structures at iron and steel plants in the Cleveland, Youngstown, Pittsburg, Johnstown and Birmingham districts. Several Erie railroad structures in which slag had been used as coarse aggregate were carefully inspected, and conclusions then and since have been of a nature favorable to the continued use of slag. As an indication of my viewpoint after this survey, I quote from my discussion of a paper on "Blast Furnace Slag as an Aggregate in Concrete," presented by W. A. Aiken at the A. S. T. M. convention in June, 1914 (A. S. T. M. Proceed., Vol. XIV):

I desire, therefore, to present for your consideration two reasons why the use of slag as a material in concrete has been so limited, and endeavor thereby to encourage discussion of a subject which must be dealt with in the concrete industry of the future.

1. Among these reasons must be recognized a lack of confidence in slag as a concrete aggregate, due rather to a limited individual experience with its use and to a very limited publication of data by those who have used it, than to specific objection to it based on personal experience with it, or on a knowledge of its behavior where used by others.

2. There exists a quite prevalent idea that slag contains elements which may act detrimentally when in combination with cement.

Let us see to what extent these are factors. They are certainly not factors as affecting construction and maintenance of masonry at the large iron and steel plants, where slag has for several years been used to the exclusion of other coarse aggregate in concrete. Here the use of slag was first limited to structures, failure in which would not be vital to the operation of the plants, but confidence once established seems to have begat confidence, and as a result there may be seen at most steel plants, slag concrete in all classes of masonry, including foundations subjected to the pounding of heavy machinery, the stoppage of which, but for an hour, might entail a loss compared with which the first cost of the foundation would be insignificant. I know of no higher compliment that can be paid to slag as a concrete aggregate than its continued use in mill construction by those who have used it most, and who are in position to know it best. This, however, should not be accepted as an argument in favor of its use in concrete for all purposes other than mill construction.

Let us make inquiry as to what is known of slag as a concrete aggregate by those who have not been associated with iron and steel mill construction. Ask of the average man interested in the concrete industry, "What do you know of slag as an aggregate in concrete?" "Have you ever used it?" "What do you think of it?" Then figure on a percentage basis and you will likely conclude that the limited use of slag in concrete for general purposes is due in part to a lack of knowledge as to its behavior where used, and to a suspicion rather than to a knowledge on the part of some that slag as an aggregate will encourage disintegration in concrete.

Each has its bearing on past usage of slag. Each, therefore, will have its influence on future usage until there is sufficient data available to enlighten those interested as to what has been done. In order to allay all fears, these data must be of such a nature as will justify confidence based on both test results and a knowledge of the behavior of slag concrete in service for several years in the various kinds of structures and classes of work.

The revenue to be had from the utilization of slag as a waste by-product, and the desire for a material of economic advantage should stimulate the publication of reliable data concerning the use of slag in concrete, but the commercial feature must not be the predominating factor, as slag must stand on its merits. Its adaptability for use in concrete road construction, in typical highway and railroad structures, and for general building purposes, must be judged from experience with its use, separately in the different classes of work.

* * *

The sum total of all comparative test results which I have been able to collect indicates that concrete in which slag was used compares favorably in strength with that in which either limestone, trap rock, or gravel was used as a coarse aggregate. Based on a careful examination of concrete in actual structure service, I am inclined to the opinion that the failures or partial failures in slag concrete have not been proportionally greater than in concrete in which other materials of accepted good quality have been used.

I have, accordingly, been looking forward with expectancy for a period of fifteen (15) years to the ultimate publication of a report such as the Committee has presented in fulfillment of the need then expressed. In the meantime, I have taken more than a casual interest in current publications of information on slag, and have followed closely the activities in the field of specification development.

The publicity which has been given since 1914 to the use of slag has brought to light many other slag concrete structures in existence at that time. When the Ulmer Building, a reinforced concrete structure in Cleveland, was torn down in 1921 after standing fifteen (15) years, I was privileged along with several other men now members of the Institute to inspect this building

during its demolition. This one instance offered particularly convincing evidence that slag concrete will protect steel against corrosion when the covering is sufficient and the concrete is carefully placed. The inspection of this building is only one of the many observations made which, collectively, afford a background of information that justifies added confidence in the suitability of slag as a concrete aggregate.

Since 1914 my close association with concrete construction on the Erie Railroad and with technical committees has afforded an opportunity to observe the durability of new and old structures of slag concrete in a climatic zone not favorable to exposed concrete structures. I have observed nothing which indicates that any mistake was made in the use of slag.

Although there are no standard tests for durability of concrete aggregates, the sodium sulphate test and the freezing and thawing test are more generally used than any others, and seemingly give promise of ultimate standardization. As brought out on page 194-5, these tests suggest a relatively high degree of durability in slag aggregate, and this is borne out by observation of its behavior in railroad ballast, macadam roads and in sewage disposal beds. These observations, in addition to those made of its behavior in concrete, lend added confidence in its durability.

Information on methods adopted for better quality control in the processing of slag has from time to time been furnished by representatives of the producing interests who have been assigned to memberships on various committees, and slag is now commercially available which is well-graded, free from foreign materials, and handled in the loading by methods calculated to minimize segregation.

In recent years, the trend in concrete practice has been toward higher sanded mixes than were common under the old-time arbitrary proportions. More consideration has been given to the bulking of moist sand, and compensation has been made for it in measuring the sand in volumetric field mixes. The adoption of the water-cement ratio principle and the broader experience with the design of concrete mixes have led to the use of higher sanded mixtures in the interest of better workability, and these have been justified by test results. The subject matter on pages 214-

15, calculated to justify the use of higher sanded mixes, is in my opinion sound. This trend has been decidedly favorable to the extended use of slag aggregate, since it overcomes in a large measure the former objections to the harshness of slag concrete in undersanded mixes. Examination of structures exposed over a long period in freezing climates indicates that outstanding examples of excellent performance are associated generally with highly sanded mixtures.

In conclusion, the fact that the technical committees of practically all the national organizations interested in construction materials have approved slag as a concrete aggregate in their specifications and reports is the one best assurance to the engineering profession that slag is a suitable coarse aggregate for use in concrete and reinforced concrete.

BY P. J. FREEMAN*

This report should receive more than the casual approval of the American Concrete Institute for the manner in which the Committee has handled its work, the completeness of the review and the method of clearing its findings to all other committees in this and similar organizations.

Blast furnace slag as a commercial commodity has been considered one of the newer ingredients in mass concrete and reinforced concrete construction in perhaps the larger portion of the United States. Its "newness" after one investigates briefly is not that at all. There are examples of concrete construction almost as old as the art of concrete making itself in which blast furnace slag was the large aggregate. The material is handicapped only by being produced and used in a relatively restricted territory and by not being advertised on a comprehensive national scale.

It is the newcomer in slag territory that is confused. To him it is a new material and something to be approached with caution and surrounded by many specification limitations and in general closely watched.

Some fifteen years ago my attention was focused on slag as an aggregate in concrete when I assumed charge of a test program

*Chief Engineer, Bureau of Tests and Specifications, Allagheny Co., Pittsburgh, Pa.

that extended over ten years. In that interval, and during the last few years, I have become well acquainted with the material. My familiarity with slag, inaugurated in the testing laboratory, has been enhanced by first hand contact with it in the field as a material which we are using in important concrete and reinforced concrete construction and also by close observation of such concrete on many inspection trips taken for various committees in this organization and other technical societies.

In the published presentation of the work of Committee 201 there has been drawn a brief but comprehensive picture of the processing, testing and uses of slag and one which has not been equalled for the more commonly used aggregates for concrete. It is particularly pleasing to note that there is so little of personal belief and opinion expressed in the report. Supporting data sustaining a comprehensive story of the subject assigned to this Committee makes the report a most valuable contribution to the technical literature of concrete construction.

A particular hobby of mine for many years has been to investigate the durability of concrete in every type of structure and in all classes of construction. In this effort to learn as much as possible about conditions affecting the durability of concrete I have seen innumerable structures in which slag was used as the coarse aggregate—probably several hundreds of them in all. Among these have been bridges, factories, warehouses, highways and possibly every example of concrete one can conceive of.

Although I have found slag structures that were not in strictly first class condition—some few of them pretty bad in fact—there has never been brought to my attention an example of troubled concrete that could be attributed directly to failure or disintegration of the slag as an aggregate. Such examples may exist, and if so, I would appreciate having them called to my attention in order that studies may be made in connection with the work of Committee 801, on Durability of Concrete.

Tests conducted under my supervision have shown that specimens of reinforcing steel embedded in samples of slag concrete for five years showed no indication of corrosion, except slight rust spots where air-pockets had formed next to the steel. This condition was common to all aggregates, gravel, traprock, granite, limestone and dolomite.

For many years I have claimed that durability should be the predominating factor in concrete construction and that the selection of both fine and coarse aggregates should be based on their ability to resist the forces tending to break down or disrupt the mass.

There has been brought to my attention a digest of something over four hundred specifications covering the uses of slag. Painstaking effort has been put forth to assemble in tabular form all of the words used in these requirements. In reading these data I was greatly interested to note that of all the words used the one which occurred most frequently was "durable." A total of 267 engineers, or groups of engineers, had imposed this requirement in their specifications.

I have seen a lot of rather severe limitations originally imposed on the material dissipated in the face of developed facts and data supported by service history until now the tendency is towards a specification in name only.

It is not my thought to give blanket approval to this material by name alone. However, when slag is adequately prepared, correctly processed, kept uncontaminated and suitably sized, it makes an excellent aggregate. If any criticism is to be made of the report of Committee 201 it is that the members who presented it did not make definite recommendations for specifications. Perhaps there were restrictions imposed on them and they might have exceeded the authority vested in the Committee, and I feel that the members should be congratulated on the scope and comprehensiveness of their work.

Discussion of Paper by Arthur Ruettgers and A. A. Whitmore

"Construction of Main Canal Lining on Kittitas Division, Yakima Reclamation Project, Washington"*

BY S. TREVOR DIBBLE†

Although situated in New Zealand at a remote distance from the main activities of the A. C. I. it is with very keen interest that descriptions of your large undertakings are followed by members here.

As that interest grows and such an excellent medium for the exchange of views is afforded by the Journal of the Institute, every member who contributes a paper for discussion by the Institute earns the thanks and gratitude of a large circle of readers who remain inarticulate through diffidence to enter the discussion for the purpose of obtaining information and replies to questions which may not be of general importance.

I wish, then, to risk that reproach and congratulate the authors, Messrs. Arthur Ruettgers and A. A. Whitmore on behalf of the members of this outpost of the Institute, for an excellent description of the efforts and methods to control the construction of this work under discussion, and also for the splendidly selected photographs which illustrate and amplify the article so well.

The authors referred to considerable transverse cracking of the lining, and found that that increased in the sections which were coated with bitumen as a curing method. May I ask whether the large amount of steel in so thin a lining, combined with the richness of the mix, would contribute largely to that defect?

*A. C. I. Journal, October, 1930, *Proceedings*, Vol. 27, p. 117.
†Registered Surveyor, Karori, Wellington, New Zealand.

It seems to me, notwithstanding that several panels collapsed through hydrostatic pressure from the back, that an adequate method of porous and tile drainage could have been provided, and the reinforcement dispensed with entirely.

With regard to transverse cracks generally in thin slabs it is, in my opinion, during the early curing periods when most of the mischief is done, and although the sprinkler idea is new to me and appears to be an excellent provision for curing in this case I would suggest that beneficial results could easily be obtained by using the burlap covering as a shade over the whole canal being concreted for fourteen days after concreting and allow the sprinklers to operate as well.

The authors drew attention to the benefits of operating two separate units on either side of the canal simultaneously for lining the sides, claiming that the spirit of competition between the gangs increased production. May I ask whether any of the bonus or differential piece rate systems for payment of labor were employed, by the contractors and if so, what were they? And were there any outstanding results noticeable from it that would be worthy of the authors' comment? Further did the tenderers for this work who appear to have competed keenly, fully understand that rigid control of quality of concrete and method of production would be literally enforced, and if so did it affect the tendering?

I should also like to know whether the rigid enforcement of quality control could always be safely entrusted to the usual job inspector, or had these men to be specially selected for their ability to accommodate their views to the new horizons in concrete work?

I may say I have found it difficult to obtain inspectors who have the necessary practical experience and knowledge of men, together with an aptitude for the learning and application of the new methods of quality control and that many petty deceptions are practiced which are hard to detect and result in false reports.

No doubt these are growing pains we feel here which your older practice of quality control will have outgrown, and I trust the authors may be able to give me an answer which will serve to guide my feet from pitfalls in the path.

AUTHORS' CLOSURE

The comments of Mr. Dibble are gratefully acknowledged by the authors, and are especially appreciated as they radiate a common interest from so distant a center of engineering activity as New Zealand.

In commenting on the transverse cracking between panel joints of the 3-in. concrete lining, Mr. Dibble asks "whether the large amount of steel in so thin a lining, combined with the richness of the mix, would contribute largely to that defect."

The authors have made a further study of the transverse cracking since their original paper was prepared and have arrived at the conclusion that the extent of such cracking can be attributed primarily to the action of the longitudinal reinforcing steel and friction of the subgrade in resisting the contraction of the concrete under falling temperatures and gradual air drying, rather than to any material fault in the fabrication, placing, or curing methods employed. One argument in favor of this conclusion is that the cracks developed progressively. On the south or shaded side slope of the lining, from 20 to 30 per cent of the cracks made their appearance during the first three weeks and the remainder during the following three weeks, while on the north or sunny slope about 60 per cent of the cracks developed during the first 3 weeks' period. Furthermore, lining placed in September was found to have approximately the same number of transverse cracks after six weeks had elapsed as lining placed in the middle of the summer when the maximum atmospheric temperatures were much higher but the daily range was nearly the same. The only evidence of crazing, indicating rapid surface drying of the green concrete, was found in several panels of Hunt-processed lining placed on July 25 when the temperature mounted to 107 deg. F. It does not appear that the richness of the mix used (if a 1:2.4:3.6 dry rodded mix may be classified as rich) has any particular bearing on the development of the transverse cracks if it be assumed, as seems reasonable, that the tensile strength of the concrete and the bond between concrete and reinforcing steel develop proportionally. If the friction of the subgrade, being a relatively minor factor, is disregarded and it is assumed that the average unit bond strength is equal to the unit

tensile strength of the concrete, it may be argued that the ultimate spacing of transverse cracks will be twice the length of steel required to develop a bond equal to the tensile strength of the concrete, or

$$L = 2\left(\frac{A}{\pi d}\right) = 2\left(\frac{\pi d^2}{4p} \times \frac{1}{\pi d}\right)$$

$$\text{or } L = \frac{d}{2p} \qquad (1)$$

in which L = ultimate spacing of cracks, A = cross-sectional area of concrete, d = diameter (or width) of bar, and p = percentage of steel.

For $d = \frac{3}{8}$ in. and $p = .31$ per cent, the values applying to the lining, we have $L = .375/.0062 = 60$ in. or 5 ft. This spacing is in reasonable accord with the average 8-ft. and 6-ft. crack spacings found in the water-cured and Hunt-processed linings respectively, especially when it is considered that additional cracks probably developed subsequent to the time the survey was made, due to increased drying shrinkage and temperature drop.

Inasmuch as equation (1) is obviously predicated on the reinforcing steel remaining intact, it becomes necessary, in investigating the possibility of restricting the cracks to the construction joints at 14 ft. intervals, to consider the question of stresses in the steel.

Assume that the steel takes its own temperature stress in addition to the stress transferred by the concrete after cracking. Then

$$f_s = f_c\,(n + 1/P)$$

in which f_s = unit tensile stress in steel, f_c = unit tensile stress in concrete, n = ratio of moduli of elasticity of steel to concrete, and P = percentage of steel.

Assume further that $n = 10$ and that the unit tensile stress in the steel, of hard grade, should not exceed 100,000 lbs. per sq. in. when the concrete has acquired a unit tensile strength of say 300 lbs. per sq. in. Then

$$100{,}000 = 300\left(10 + \frac{1}{P}\right)$$

$$\textit{and } P = .0031$$

Substituting this value of P in equation (1) we have

$$L = \frac{d}{2\ (.0031)} = 162\ d.$$

If $d = \frac{1}{4}$ in. ($5\frac{1}{4}$ in. spacing), $L = 41$ in. $= 3$ ft.-5 in.
If $d = \frac{3}{8}$ in. (12 in. spacing), $L = 61$ in. $= 5$ ft.-1 in.
If $d = \frac{1}{2}$ in. ($21\frac{1}{4}$ in. spacing), $L = 81$ in. $= 6$ ft.-9 in.

It appears from the foregoing that it is impracticable to design a 3-in. reinforced concrete lining, under the conditions mentioned, to insure an ultimate uniform distribution of transverse cracks at 14 ft. intervals. If steel in the amount of .31 per cent is used, intermediate cracks must be anticipated or the panel length reduced to not over 7 ft. Or if the percentage of steel is reduced to conform to a construction joint spacing greater than 7 ft., ultimate failure of the steel and non-uniform distribution and width of joint openings must be expected.

Mr. Dibble expressed the opinion "that an adequate method of porous and tile drainage could have been provided, and the reinforcement dispensed with entirely." The nature of the country traversed and the variation in soil conditions made the problem of draining the lining one of the most difficult ever encountered by the Bureau of Reclamation and the matter was given very careful attention. With a few minor exceptions the drainage facilities provided appear to have proven adequate and successful to a degree surpassing all expectations.

While the use of plain concrete would undoubtedly have eliminated the intermediate transverse cracks, or at least materially reduced their number, the authors do not look upon omission of the reinforcement with favor, particularly in a region subject to severe frost and unfavorable drainage conditions. The intermediate cracks are not regarded as a serious menace to the integrity of the lining and it is believed that the additional expense for reinforcement (about 10 per cent of the cost of the lining) will prove a judicious investment in providing continuity and increased strength as security against heaving, uneven settlement, undermining, earth or water pressure, flotation, impact from falling rocks, serious washouts, or other conditions which might develop.

With regard to the spirit of competition displayed by the two lining-machine operating crews working simultaneously on both

side slopes of the canal, it may be said that the attitude manifested was apparently spontaneous and not induced by any piecework or bonus system of payment, so far as the authors are aware. The rate of production was held within bounds by the inspector's insistence on proper placement and compacting of the concrete, as well as by the desire of the foreman to avoid reworking and extra finishing.

Although very little trouble was experienced with concrete inspectors, it is the opinion of the authors that rigid enforcement of quality of concrete cannot be safely entrusted to the usual job inspector without apprizing him in all seriousness of the requirements, providing him with the needed implements and information in an understandable and convenient workable form, supplying supervision commensurate with his reactions, and above all instilling an interest and pride in his work. The inspectors were selected from a civil service register of applicants meeting the minimum educational and experience requirements. There were, of course, differences in type and character of the men, but on the whole the inspectors employed proved to be conscientious, trustworthy and efficient. A good many, in fact, developed into enthusiastic control advocates. What few deceptions were practiced were soon detected and obliterated, if necessary by peremptory discharge. It goes almost without saying that wholehearted support and positive backing by the engineer in charge is essential to the complete success of any attempt at rigid concrete control.

In closing, the authors desire to call attention to one feature of the lining not mentioned in Mr. Dibble's discussion but which may be of general interest, because of its practical significance. The original paper stated, in connection with the final adoption of the plain butt type of panel joint, that the use of the edging tool for grooving the top surface of the concrete at the joint was early discontinued upon finding that the groove was not required to insure the development of the crack. Information just received, however, is to the effect that some appreciable spalling has occurred along sharp edges of the cracks and that the edging tool, which provides rounded shoulders, is again being utilized on lining work.

Reinforced Concrete Column Investigation*

Second Progress Report of Committee 105

Committee 105 presented its first progress report at the 1930 convention of the American Concrete Institute in New Orleans. This report was published in the April, 1930 Journal, and is found on pages 601 to 615 of the 1930 *Proceedings*. It contained the program of tests as laid out by the committee, and the results of some of the preliminary investigations on the materials to be used. Since that time, three committee meetings have been held and the investigational work is well under way. In carrying out the tests slight modifications of the program have been found necessary, but in the main the original program has been followed.

At the University of Illinois all the specimens for this program have been made. At Lehigh University all the specimens except the greater portion of those in Series 4 have been made. At both universities many of the specimens have been tested and the work of interpreting the data is under way.

At a meeting of the committee held in December, 1930, at the Bureau of Standards, Washington, D. C., it was decided unanimously, that these participating laboratories, the University of Illinois, and Lehigh University, should be authorized to submit their own progress reports of the tests. Since the tests for Series 1 and 2 had already been completed, it was found advisable to publish the results for these two series in the February Journal of the Institute. These laboratory reports follow the report of the committee. They have been prepared independently by each laboratory, and are intended to present as concisely as possible, the results of the tests to date for future use of the committee and the members of the Institute. These reports do not bind the committee to the expression of any opinion as to their significance.

*For presentation and discussion at the 27th Annual Convention, Milwaukee, Feb. 24-26, 1931.

The work of Series 3 has also been carried far enough to warrant a similar progress report, but it was believed desirable to delay this report slightly in order to include as late data as possible. In conformity with this program it is expected that a further progress report covering Series 3 will be available in preprint at the Milwaukee convention in February.

COMMITTEE 105—REINFORCED CONCRETE COLUMN INVESTIGATION

W. A. SLATER, *Chairman*
P. H. BATES
R. L. BERTIN
F. E. RICHART
R. D. SNODGRASS
W. S. THOMSON
F. R. McMILLAN, *ex-officio*
as Chairman Dept. 100, Research

Readers are referred to the JOURNAL *for March, 1931, for a further progress report, and to the* JOURNAL *for June, 1931, for discussion which may develop. Such discussion should reach the Secretary by May 1, 1931.*

First Progress Report on Column Tests at Lehigh University

BY W. A. SLATER* AND INGE LYSE†

INTRODUCTION

The tentative program and the results of the preliminary tests of the concrete for the reinforced concrete column investigation being carried out under the sponsorship of the American Concrete Institute was published in the Journal for April, 1930[1]. The parts of the report presented herein have been prepared by those having charge of the tests at Lehigh University. The authors accept the responsibility for the statements made. An effort has been made to avoid all generalizations but to present the data as fully as possible at the present time and to point out the important relations indicated in Series 1 and 2.

SERIES 1. STUDY OF END CONDITIONS

1. *Purpose and Program*—The purpose of Series 1 of this investigation was to study the effect of different end conditions on the strength and other properties of the columns, and from the results obtained decide upon the end condition to be used in the remaining series. This series was therefore completed before the others were begun.

The columns included in this series had a diameter of 8 in., an overall length, or shaft length, of 60 in., and were tested at an age of 28 days. All columns contained eight ½-in. square longitudinal reinforcing bars and a $\frac{3}{16}$-in. wire spiral of 1.35-in. pitch. This corresponds to about 4 per cent of longitudinal, and 1.2 per cent of lateral reinforcement. The concrete was designed for a strength of 3500 lb. per sq. in. at 28 days as determined on 6 by 12-in. cylinders at the time of testing the columns.

*Director, Fritz Engineering Laboratory, Lehigh University. Chairman, Committee 105.
†Assistant Engineer, Portland Cement Association, In immediate charge of column tests at Lehigh University.
[1]*A. C. I. Proc.*, Vol. 26, p. 601.

Series 1 included three types of end condition. In the first type there were no capitals, and the ends of the longitudinal reinforcing bars were flush with the ends of the column. The second type had 14-in. diameter capitals. The third type had dowels and no capitals. Two groups of columns of the third type were tested. One group had 13-in. dowels which lapped over the reinforcing bars for a length of 20 diameters. The other group had 18-in. dowels providing for a 30-diameter lap. There was the same number of dowels as of the longitudinal bars, and both were of the same grade of steel. Two columns were included in each group.

2. *The Concrete*—The concrete used in this series was a 1:2.4:3.6 mix by weight. The cement content was 5.4 sacks per cu. yd. of concrete. The nominal water content was 40.5 gal. per cu. yd. of concrete or 7.5 gal per sack of cement. The concrete was mixed in a kettle-shaped mixer of about 2.5 cu. ft. capacity. The dry materials were mixed for one minute. Water was then added and the mixing was continued for three minutes more. Six 6 by 12-in. cylinders were made with each column, three for test at the age of 28 days, and three for test at 56 days.

Table 1 gives information on the properties of the concrete used in this series.

3. *The Reinforcement*—The longitudinal reinforcing bars were donated by the Bethlehem Steel Company of Bethlehem, Pennsylvania. These were plain bars having an average yield-point stress of 48,000 lb. per sq. in. and an average ultimate strength of 76,850 lb. per sq. in. The test results of five coupons taken from five different bars show a good degree of uniformity.

Coupon Number	1	2	3	4	5	Av.
Yield-Point Stress*	47,520	48,800	48,040	47,560	48,120	48,000
Ultimate Stress*	76,680	77,000	76,760	76,800	77,000	76,850

*Stresses given in pounds per square inch.

The lateral reinforcement was donated by the American Steel and Wire Company of Chicago, Illinois, and fabricated into spirals by the American System of Reinforcing. It consisted of $\frac{3}{16}$-in. wire spirals having a pitch of 1.35 in. A definite yield point could not be detected for the spiral reinforcement. Therefore only ultimate strength was obtained.

Coupon Number	1	2	3	4	5	6	7	8	Av.
Strength lb. lb. per sq. in.	No Test	89,340	83,570	85,730	83,930	83,570	87,170	77,450	84,390

It will be noted that the lateral reinforcement was less uniform but about 10 per cent higher in average strength than the longitudinal reinforcement.

4. *Fabrication of Reinforcement*—All the reinforcement for each column was assembled into a unit before being placed in the mold. Four of the longitudinal bars had six ½-in. steel cubes welded to the side so as to form a continuous row of five 10-in. gage lengths. The end cubes were placed five inches from the ends of the column or ends of the shaft.

The spirals had three spacers which were placed approximately 120 degrees apart. Eight longitudinal bars were fastened with wires to the inside periphery of the spirals approximately 45 degrees apart.

5. *Making Columns*—In Group 1 the longitudinal bars were flush with the ends of the column. This provided for transferring the load from the machine directly upon both the concrete and the steel. In making all columns, the concrete was filled to the top of the molds. A settlement of about $\frac{1}{16}$-in. took place during the setting, due to the consolidation of the mass. Neat cement paste having a water-cement ratio of 0.3 by weight, re-mixed

TABLE 1—PROPERTIES OF CONCRETE

Group No.	End Condition	Average Slump In.	28-Day Strength of Control Cylinders lb. per sq. in.								Average for Group
			Cylinders for Column A				Cylinders for Column B				
			1	2	3	Av.	1	2	3	Av.	
1	Plain	4¾	3075	2920	2770	2920	3510	3340	3200	3350	3140
2	Capital	4¼	3220	3030	3260	3170	3380	3740	3140	3420	3300
3	(20-dia. splice)	4½	3460	3560	3500	3510	3460	3620	3440	3510	3510
4	(30-dia. splice)	3½	3260	*	2980	3120	3340	3410	3280	3340	3230

*Broken Cap

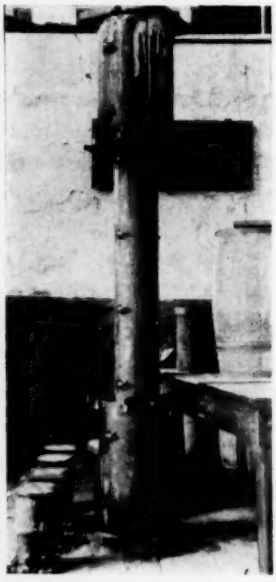


FIG. 1—MAKING COLUMNS HAVING CAPITALS

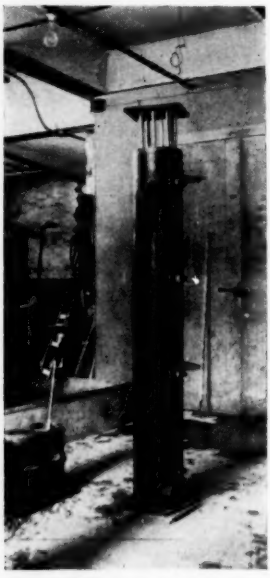


FIG. 2—MAKING COLUMNS HAVING DOWELS

at four hours, was used as capping material. Planed steel plates used for the capping were worked down upon the ends of the longitudinal bars, bringing them flush with the surface of the concrete. The caps were usually about $\frac{1}{16}$-in. thick.

The columns of Group 2 had a capital at each end. The shaft had the dimensions of the plain columns. The capitals were 18 in. long and 14 in. in diameter. The longitudinal bars extended a distance of 30 diameters, or 15 in. into the capital. The spiral extended six inches into the capital. Wire hoops were placed in each capital to prevent its falling apart in case of splitting during the testing. The form used for making the columns of Group 2 is shown in Fig. 1.

The columns of Group 3 had dowels of such a length as to form a 20-diameter splice. The longitudinal reinforcing bars extended to within three inches of each end, and the dowels were flush with the ends. Group 4 was of the same type as Group 3 except

that a 30-diameter splice was used. Screws which passed through the end plates and into the ends of the dowels, held the dowels flush with the surface of the concrete and kept them in position during the placing. Fig. 2 shows the placing of the top plate on a column having dowels. The mold was filled to the top and the dowels were then forced into the concrete until the plate rested on the top of the mold. At the time of capping, the screws were removed and the end plate lifted. The capping was done in the ordinary manner.

In making the columns, the concrete was placed continuously and was rodded into place with ½-in. steel rods. The mold was tapped with a hammer in order to consolidate the mass. At the age of 24 hours the molds were removed, and a ¼-in. diameter by ½-in. steel plug was set in the concrete with neat cement paste beside each lug on the longitudinal bar. The columns were then given three coats of linseed oil and placed in the moist room. One day before testing, each column was removed from the moist room and gage holes drilled in all the plugs and lugs. Five continuous 10-in. gage lengths were thus obtained on both the steel and the concrete on elements 90 degrees apart, around the column. Elements 1 and 3 were at opposite ends of one diameter, and 2 and 4 at opposite ends of the other.

6. *Testing*—The columns were cured in the moist room until they reached an age of 28 days, and then were tested to failure in an 800,000-lb. vertical screw testing machine. For the columns without capitals, an eight-inch spherical bearing block was placed between the head of the machine and the top of the column. The columns with capitals required a larger spherical bearing block and a 17-in. block was used. It was so heavy that it was placed at the bottom instead of the top. The column was plumbed by means of plumb bobs.

Before each column with plain ends was loaded, a complete set of strain-gage readings was taken. For the columns with capitals, a load of 200 lb. was applied before the initial readings were taken. With the spherical bearing block at the base of the column, this load was necessary to hold the column in line.

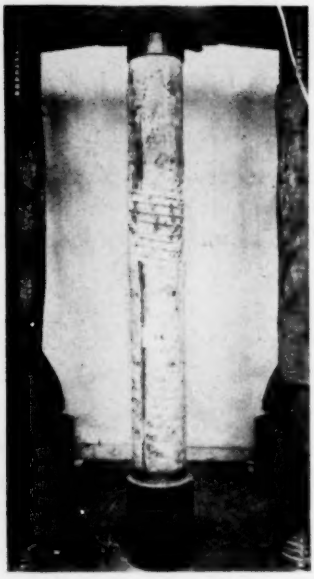


FIG. 3—COLUMN A OF GROUP 1 AFTER TEST

FIG. 4—COLUMN B OF GROUP 2 AFTER TEST

Five 10-in. gage lengths were observed on four rows of lugs on the steel bars with corresponding readings on the plugs in the concrete. This makes a total of 20 gage lengths on the steel, and 20 in the concrete. Each gage length was observed twice, so that 80 observations were made for each set of readings.

The testing crew consisted of two observers and a recorder. The observers took readings on opposite ends of the same diameter simultaneously. The order used in taking the measurements was as follows: Readings were taken from top to bottom on the steel in rows 1 and 3, and then from bottom to top on the concrete. The same order was used for readings on rows 2 and 4. A second set of readings was taken in a similar manner. Ordinarily the maximum variation between the readings of the two sets was 0.0002 in. as read on the dial of the Whittemore strain gage used. If a larger variation occurred a check reading was taken.

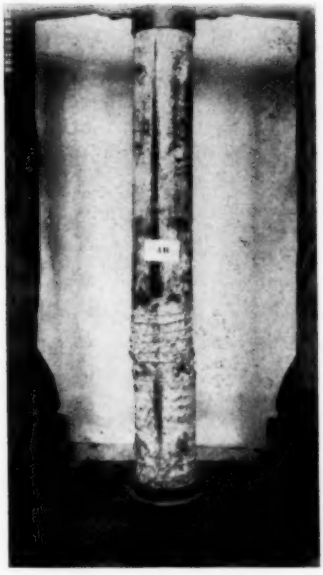


FIG. 5—COLUMN B OF GROUP 3 AFTER TEST

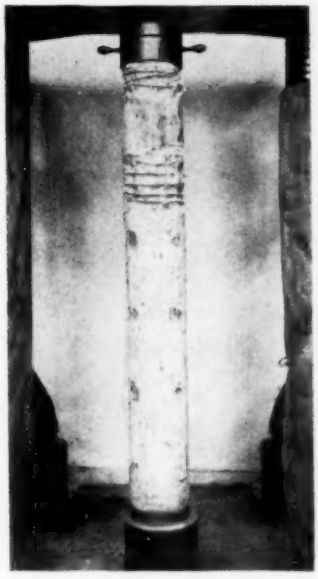


FIG. 6—COLUMN A OF GROUP 4 AFTER TEST

The load was applied in increments of approximately 25,000 lb. A complete set of observations was made for each load. The beam of the machine was rebalanced after the observations so that the load, both at beginning and end of observations, was recorded. This procedure was followed until the deformations went beyond the range of the strain gage on some of the gage lengths.

The time required for each loading, that is, from one application of the load to the next, varied. After some experience, the time for each set of observations ranged from seven to eight minutes.

7. *Strength of Columns*—All columns had 4 per cent of longitudinal reinforcement and 1.2 per cent of spiral reinforcement. The concrete was of the same mix and water-cement ratio throughout the series. Due to minor variations, however, slight differences in strengths were obtained. The test results of the control cylinders are given in section 2. The major difference in

FIG. 7—DETAILS OF FAILURE OF COLUMNS HAVING DOWELS

strength of the columns is, therefore, attributed to the end condition. Table 2 gives the maximum strengths of the columns having different end conditions. Figs. 3 to 7 inclusive, show the types of failure for the groups. The columns of Group 1, shown in Fig. 3, failed near the center. The columns with capitals, shown in Fig. 4, failed about ten inches below the capital. The columns with dowels failed at the ends of the dowels, either at the top or bottom of the column, as illustrated in Figs. 5 and 6. Fig. 7 shows the details of the failure of columns having dowels. After removal of the load a gap was generally found between the end of the reinforcing bar or dowel and the concrete. This is illustrated in Fig. 7. At the ends of the columns also the bars had been pulled back into the concrete slightly. This was true in the columns with continuous bars as well as in those with dowels. It appears that for some reason the concrete elongated more than did the bars on removal of the load. The cause of this phenomenon has not been explained, but it may have been

TABLE 2—STRENGTH OF COLUMNS AND THEIR CONTROL CYLINDERS

Group No.	Maximum Load, lb.			Strength of Control Cylinders lb. per sq. in.		
	Col. A	Col. B	Av.	Col. A	Col. B	Av.
1	252,800	265,000	258,900	2,920	3,350	3,140
2	260,000	251,000	255,500	3,170	3,420	3,300
3	218,600	245,400	232,000	3,510	3,510	3,510
4	231,000	231,000	231,000	3,120	3,340	3,230

due partly to the buckling of the bars as shown in Fig 3 and partly to the assistance of the spiral reinforcement in maintaining the integrity of the concrete.

In order to make a comparative study of the effect of the end condition on the strength of the columns, the variation in the strength of the concrete must be taken into account.

It is shown in Series 2 that the load carried by the plain columns was very nearly 85 per cent of the strength of the control cylinders regardless of the strength of the concrete. Assuming that the concrete in the core of a spirally reinforced column will carry the same amount of load as in a plain column, the remaining load may be attributed to the presence of the steel, both spiral and longitudinal. In computing the load carried by the concrete the area used in the core area specified by the American Concrete Institute Building Code, that is, "the area within the outer circumference of the spiral hooping." As the outer diameter of the spirals was 7.75 in. and as the sectional area of the longitudinal bars used was 2 sq. in., the net core area of the concrete was:

$$\frac{\pi}{4}(7.75)^2 - 2 = 45.1 \text{ sq. in.}$$

The load attributed to the concrete was then 0.85 x 45.1 f'_c, where f'_c = unit strength of the control cylinders. Table 3 gives the computed division of load between concrete and reinforcement at the time of failure. The values given in this table have been plotted in Fig. 8. The upper line in this figure represents the proportionate total loads, and the lower line represents the proportionate loads attributed to the reinforcement in the different groups. The fact that the curve for load carried by the steel

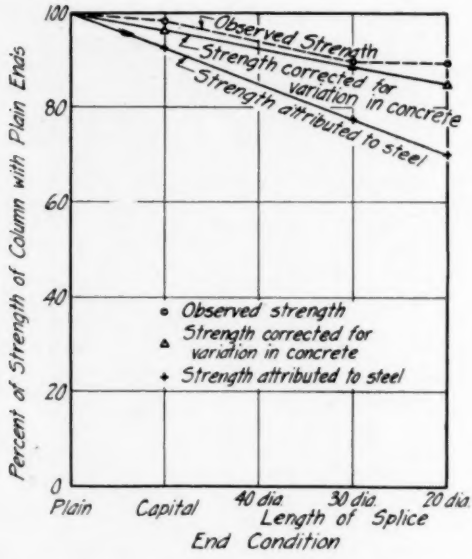


FIG. 8—RELATION BETWEEN END CONDITION OF COLUMN AND STRENGTH

is a straight line is due to a suitable selection of horizontal scale. Fig. 8 indicates that in order to obtain the same effect due to the reinforcement in a column with spliced ends, as in a column with bars flush with the ends, the length of the splice would have to be 60 times the diameter of the bars. The capitals apparently

TABLE 3—DIVISION OF LOAD BETWEEN CONCRETE AND REINFORCEMENT

Group No.	End Condition	Maximum Load		Load Attributed to Concrete	Load Attributed to Steel	
		lb.	per cent	lb.	lb.	per cent
1	Plain	258,900	100	120,500	138,400	100
2	Capital	255,500	98.5	126,800	128,700	93.0
3	(20-dia. splice)	232,000	89.5	135,000	97,000	70.0
4	(30-dia. splice)	231,000	89.2	124,000	107,000	77.4

TABLE 4—COLUMN STRENGTH CORRECTED FOR VARIATION IN CONCRETE STRENGTH

Group No.	Load Attributed to		Total Load	
	Concrete lb.	Steel lb.	lb.	per cent
1	134,500	138,400	272,900	100
2	134,500	128,700	262,200	96.1
3	134,500	97,000	231,500	84.8
4	134,500	107,000	241,500	88.5

gave the equivalent of a splice of 50 diameters.

The load carried by the steel would probably have been no different if the cylinder strength had been exactly 3500 lb. per sq. in. for all groups. Assuming that this is true the strength of the reinforced column for 3500-lb. concrete has been estimated by adding to the load carried by the reinforcement 85 per cent of the assumed cylinder strength of 3500 lb. per sq. in. The computations are shown in Table 4 and the column strengths corrected in this manner are shown in Fig. 8.

Fig. 8 indicates that the columns with the longitudinal steel flush with the ends of the column, developed higher strength than the columns with capitals or spliced reinforcement. The comparative ease of making this type of column makes it less expensive than the columns having dowels and much less expensive than the columns with capitals.

8.—*Deformations*—In order to study the relation between deformations at different elevations, the average deformations for each group of columns have been plotted in Figs. 9 to 12 inclusive.

Fig. 9 shows the average deformations for the columns of Group 1. This figure illustrates that at low loads the uniformity in the elastic properties at different elevations was good. For loads of 100,000 lb. or less, the deformations were practically equal at all elevations of the column. For higher loads, however, the deformations at the center elevation of the column were considerably larger than those at the ends. The most satisfactory explanation of this phenomenon seems to be the possibility of lack of homogeneity of the concrete. At low loads the steel

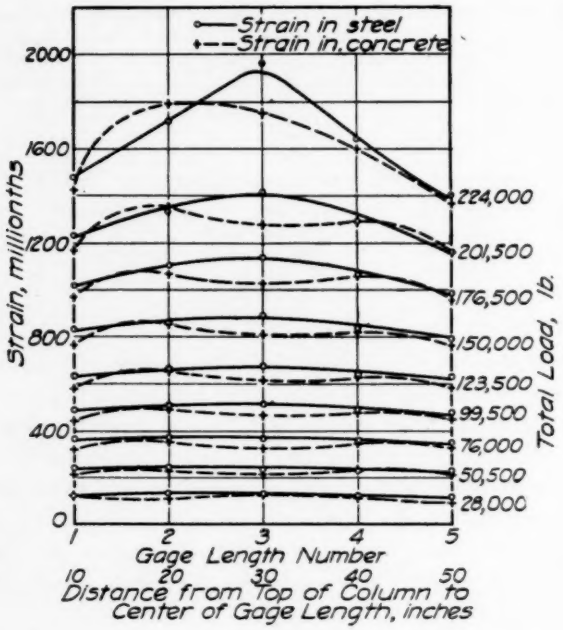


FIG. 9—DEFORMATION AT DIFFERENT ELEVATIONS FOR COLUMNS OF GROUP 1

and the concrete deformed together. At higher loads this equality of deformations continued near the ends, but near the center the deformations in the steel exceeded those in the concrete. The excess of deformation in the steel over that in the concrete increased as the load increased and failure occurred at the center as shown in Fig. 3.

Fig. 10 shows the average deformations of the columns having capitals. It is seen from the figure that the deformation of the steel was nearly equal to that of the concrete for all loads. It is noted, also, that the deformations at different elevations were nearly equal at low load, while for high loads the deformation was much greater at the top than at the bottom.

Fig. 4 shows that the failure occurred near the top of the shaft as would be expected from the behavior described. Slightly

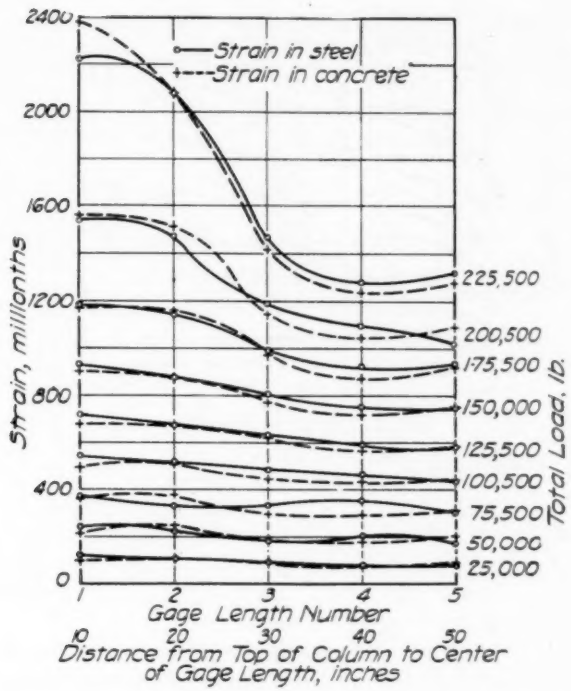


FIG. 10—DEFORMATION AT DIFFERENT ELEVATIONS FOR COLUMNS OF GROUP 2

weaker concrete just below the capital, may have caused the deformations near the top to be larger than those near the bottom. A noticeable eccentricity in the loading (see Fig. 18) probably added to the stresses at the top of the shaft.

Fig. 11 shows the deformations at different elevations for the columns of Group 3. These columns had dowels which provided for a 20-diameter splice. As before, the deformations were nearly equal at different elevations for low loads. In general, the steel and concrete deformed together until a load of about 50 per cent of the maximum strength of the column was reached. At this load the concrete in the bottom section deformed much more than did the steel, indicating that slipping of the bars must have taken place. As the load increased, the slipping between steel and

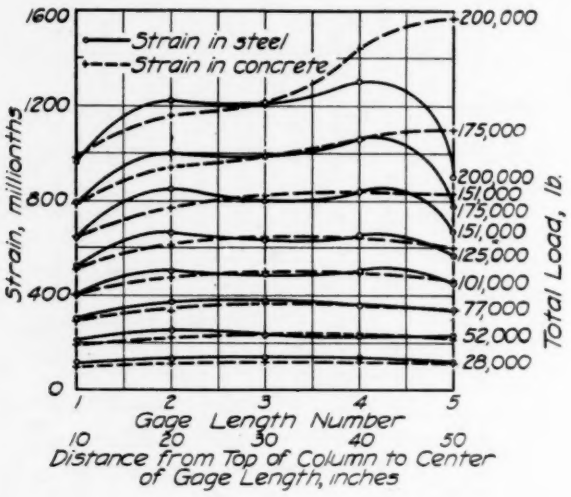


FIG. 11—DEFORMATION AT DIFFERENT ELEVATIONS FOR COLUMNS OF GROUP 3

concrete became more pronounced, and near maximum load it was noted that the concrete had moved away from the lugs on the steel forming a perceptible gap. The concrete spalled at the ends of the dowels and at the ends of the longitudinal bars at practically the same load, indicating that dowels and bars began to slip simultaneously.

Fig. 12 shows the deformations at different elevations for the columns of Group 4. These columns had dowels which lapped over the longitudinal reinforcing bars for a distance of 30 diameters. The figure indicates that the concrete and the steel deformed together throughout the test, for points at the center and below, and also for points above the center, until about 75 per cent of the maximum load had been reached. At this load a slip occurred between steel and concrete near the top of the column, and the deformations of the concrete there became much larger than those of the steel. The concrete deformed very fast while the stress in the steel did not increase materially. The spalling of the columns occurred at the ends of the longitudinal bars and at the ends of the dowels as in Group 3. The failure

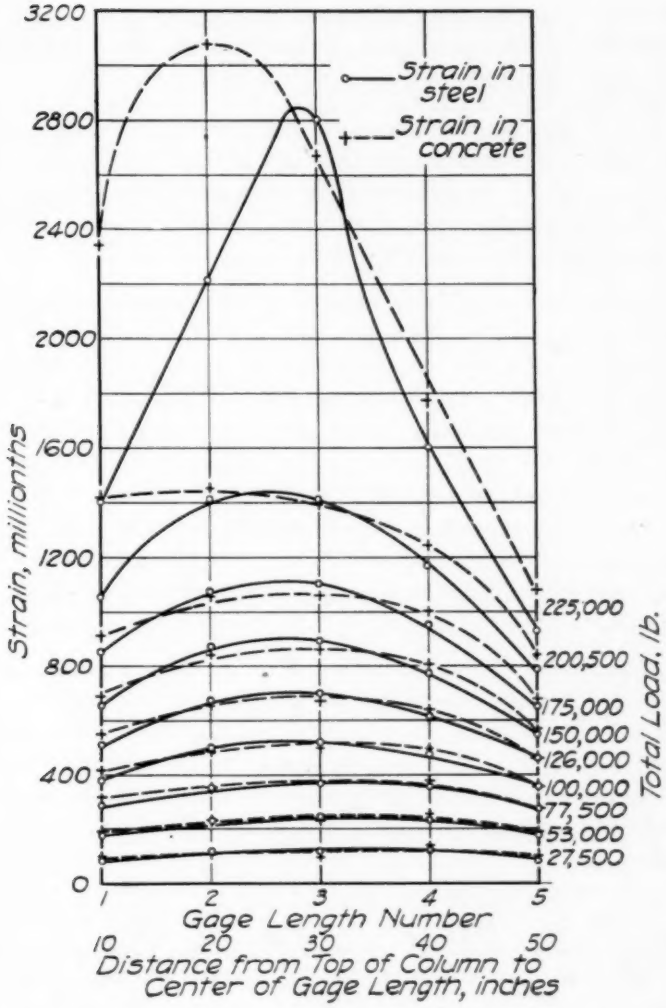


FIG. 12—DEFORMATION AT DIFFERENT ELEVATIONS FOR COLUMNS OF GROUP 4

of these columns was very similar to that of Group 3, except that the columns of Group 4 failed at the top while those of Group 3 failed at the bottom.

For both groups of columns having dowels, the deformations at all loads were smaller near the ends than at the center. This probably was due to the fact that on account of the lapping of the dowels the percentage of steel at the end section was twice the percentage at the center section.

9. *Load Distribution*—In Figs. 13 to 16, inclusive, the average deformations are plotted for the different loads on the columns. In addition, the loads corresponding to the strains in the steel are represented by a broken straight line. This line is carried to a strain of 1600 millionths, corresponding to a yield-point stress of 48,000 lb. per sq. in. within which the modulus of elasticity of the steel was assumed as 30,000,000 lb. per sq. in. No determination of the modulus of elasticity of steel was made in this series of tests. The assumed division of the maximum load carried by the columns is shown on the right-hand side of the diagrams.

Fig. 13 shows the deformation diagram for the columns of Group 1. The assumption has been made that the concrete, the longitudinal steel, and the spiral have jointly contributed to the column strength. The load attributed to the concrete at the maximum load is the product of 85 per cent of the average cylinder strength and the core area of the column, that is, 0.85 x 3140 x 45.1, or 120,500 lb. The load attributed to the longitudinal steel is the product of the yield-point strength and the area of the steel, that is, 48,000 x 2, or 96,000 lb. The sum of the loads attributed to the concrete and to the longitudinal steel (120,500 + 96,000 or 216,500 lb.) has been subtracted from the column strength (258,900 lb.). This difference (258,900 — 216,500 = 42,400 lb.) has been attributed to the spiral.

According to the above assumption, the load attributed to one per cent of longitudinal steel is $\frac{96000,}{4}$ or 24,000 lb. The 1.2 per cent of spiral apparently contributed 42,400 lb., indicating that one per cent of spiral was as effective as 1.47 per

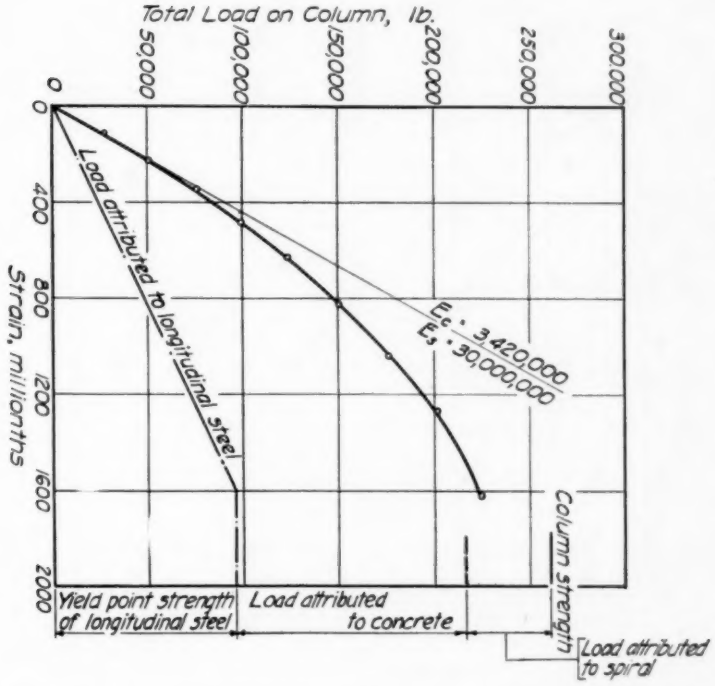


FIG. 13—DEFORMATION DIAGRAM FOR COLUMNS OF GROUP 1

cent of longitudinal reinforcement for the columns of Group 1.

The deformations at top and bottom gage lengths for the columns of Group 2 and the division of load, are shown in Fig. 14. The total load on the column has been distributed between the concrete, the longitudinal steel, and the spiral. It is noted that the load attributed to the 1.2 per cent of spiral is 32,700 lb., or 1.14 times as much as for an equal percentage of longitudinal reinforcement.

Fig. 15 shows the deformation diagram and the division of the load for the columns of Group 3. Here the sum of the yield-point strength of the steel and the load attributed to the concrete is almost equal to the maximum strength of the column. The spiral did not seem to add anything to the strength of the columns,

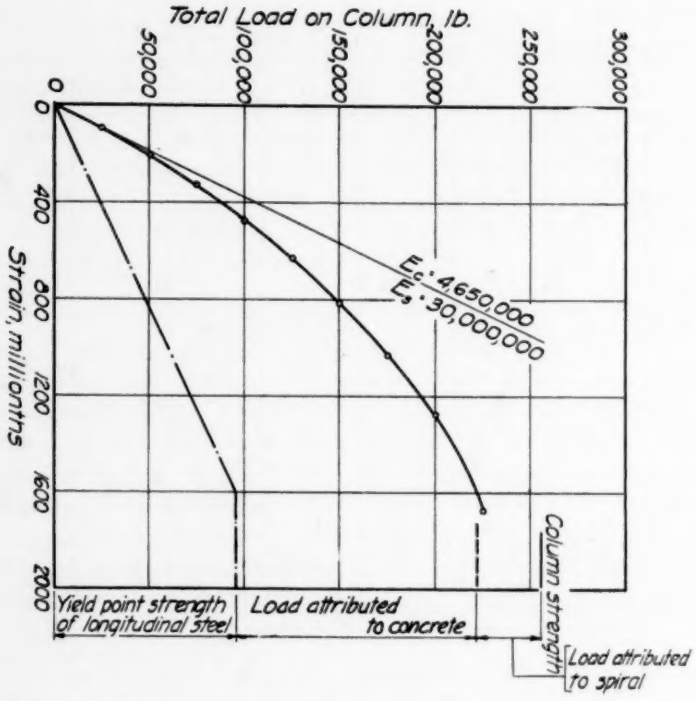


FIG. 14—DEFORMATION DIAGRAM FOR COLUMNS OF GROUP 2

probably because failure was precipitated by the slipping of the dowels before the spirals could come into action.

The deformation diagram and the division of the load for the columns of Group 4 are shown in Fig. 16. In this case the strength of the column was about 11,000 lb. greater than the sum of the yield-point strength of the steel and the load attributed to the concrete. This 11,000 lb. may be attributed to the spiral, but the spiral probably was not fully effective because of slipping of the dowels. It is of interest, however, to point out the fact that the 30-diameter lap was more effective than the 20-diameter lap.

10. *Eccentricity of Loading*—If a plane section remains plane during the loading, the sum of the strains ϵ_1 and ϵ_3 measured at the ends of one diameter, should be equal to the sum of the

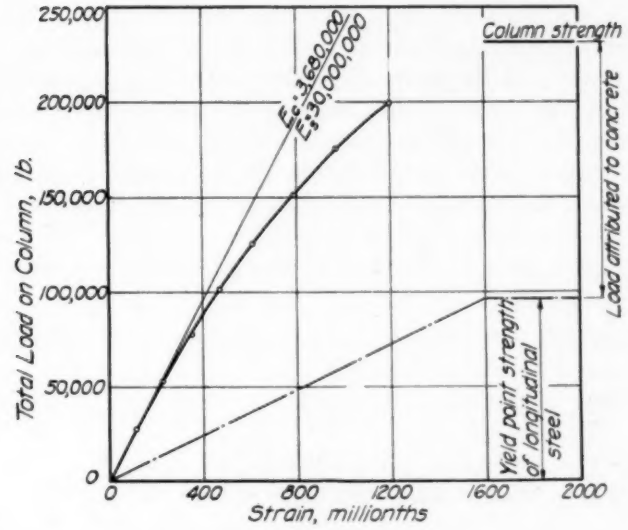


FIG. 15—DEFORMATION DIAGRAM FOR COLUMNS OF GROUP 3

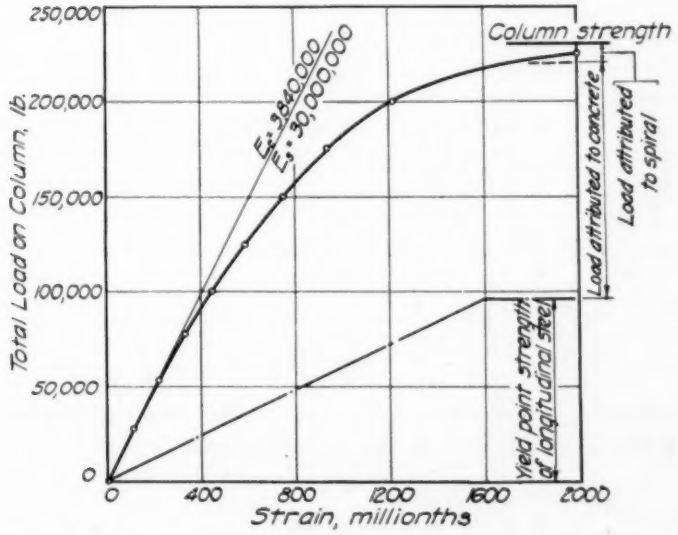


FIG. 16—DEFORMATION DIAGRAM FOR COLUMNS OF GROUP 4

strains ϵ_2 and ϵ_4 at the ends of another diameter. This relation was found to hold approximately true for all the columns. The strain volume is, therefore, a cylinder with ends parallel in the case of centric loading, and non-parallel for eccentric loading. Assuming that the stress is proportional to the strain, this relation may be used to determine the size and shape of the stress cylinder and the distance of the center of pressure from the center of the section, that is, the eccentricity of loading. The center of pressure must pass through the center of gravity of the stress cylinder in order to establish equilibrium.

The following method was used in determining the position of the center of pressure. (See Fig. 17).

The volume of the stress cylinder is:

$$V = \pi r^2 \epsilon E$$

where r = the radius of the cylinder

ϵ = the average strain, that is, $\frac{1}{4}(\epsilon_1 + \epsilon_2 + \epsilon_3 + \epsilon_4)$ and

E = modulus of elasticity

The bending strain, ϵ_b is: $\epsilon_b = r\tan\Theta$, where Θ is the angle between the end planes of the stress cylinder

The bending moment is:

$$M = \frac{\epsilon_b}{c} EI$$

where I = moment of inertia of the cross section

and c = distance from the neutral axis to the point at which ϵ_b is measured

Now

$$I = \frac{\pi r^4}{4} \qquad \text{and } c = r$$

Then

$$M = \frac{r\tan\theta}{r} \; E\frac{\pi r^4}{4} = \frac{\pi r^4}{4} E\tan\theta$$

The distance e from the neutral axis to the center of gravity of the stress cylinder, that is, the eccentricity is:

$$e = \frac{M}{V} = \frac{\frac{\pi r^4}{4} E\tan\theta}{\pi r^2 \quad \epsilon E} = \frac{1}{4}\frac{r^2}{\epsilon}\tan\theta$$

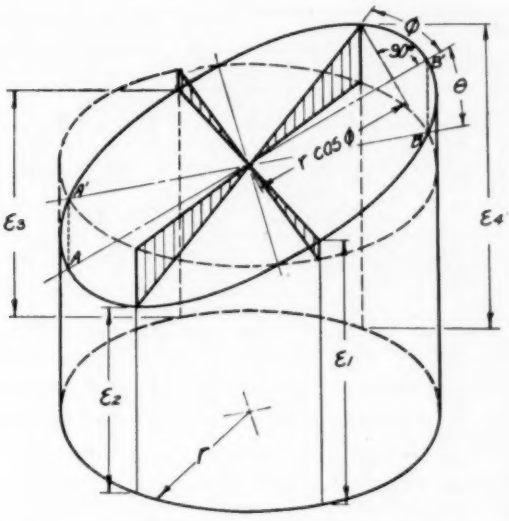


FIG. 17—ILLUSTRATION OF STRAINS USED IN DETERMINING ECCENTRICITY

From the illustration:

$$tan\theta = \frac{\epsilon_4 - \epsilon_2}{2rcos\phi} = \frac{\epsilon_1 - \epsilon_3}{2rsin\phi}$$

where ϕ is the angle between a plane passing through elements 2 and 4 and a vertical plane passing through the line of maximum and minimum stresses. The center of gravity of the stress cylinder will lie in the plane passing through the maximum and minimum strains at a distance, e, from the center line of the cylinder.

The angle ϕ is determined from:

$$\frac{\epsilon_4 - \epsilon_2}{2rcos\phi} = \frac{\epsilon_1 - \epsilon_3}{2rsin\phi}$$

$$\text{or} \quad \frac{2rsin\phi}{2rcos\phi} = \frac{\epsilon_1 - \epsilon_3}{\epsilon_4 - \epsilon_2} = tan\phi$$

$$e = \tfrac{1}{4}\frac{r^2}{\epsilon} tan\phi = \tfrac{1}{4}\frac{r^2}{\epsilon}\frac{\epsilon_4 - \epsilon_2}{2rcos\phi} = \frac{r}{8\epsilon}\frac{\epsilon_4 - \epsilon_2}{cos\phi}$$

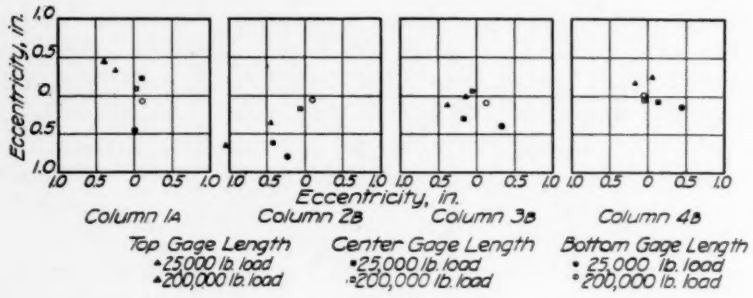


FIG. 18—ECCENTRICITY OF LOAD ON COLUMNS IN SERIES 1

Therefore the eccentricity is given by the formula:

$$tan\phi = \frac{\epsilon_1 - \epsilon_3}{\epsilon_4 - \epsilon_2} \quad \text{and} \quad e = \frac{r}{8\epsilon} \frac{\epsilon_4 - \epsilon_2}{cos\phi}$$

Thus the center of pressure may be easily determined from the readings obtained on the four elements 1, 2, 3, and 4. In Fig. 18 the center of pressure is given for top, center, and bottom elevations for loads of 25,000 and 200,000 lb. respectively.

This figure shows that the eccentricity was in all cases considerably less for the load of 200,000 lb. than for 25,000 lb. Also, it was generally less for the center and the bottom than for the top of the column. It may be shown that the ratio of bending stress to total stress is equal to the ratio $\frac{4e,}{r}$ that is, four times the ratio of the eccentricity to the radius of the column. For the columns, without capitals this ratio, $\frac{4e,}{r}$ was always less than $\frac{1}{2}$. For the 200,000-lb. load it was generally much less. For the columns with capitals the ratio, $\frac{4e,}{r}$ was greater than $\frac{1}{2}$ in all cases for the load of 25,000 lb. and at the top it exceeded 1, that is, the bending stress in this case was greater than the average stress, $\frac{P.}{A}$ However, for the load of 200,000 lb. the bending stress had been reduced to about one-half the average stress. For all

the other columns at the load of 200,000 lb. the bending stress was generally less than 20 per cent of the average stress $\frac{P}{A}$. The foregoing analysis has been based upon an assumed constant modulus of elasticity. The fact that in general the modulus is not constant will operate to reduce somewhat the eccentricity and the ratio of bending to average stress.

11. *Summary*—The columns having longitudinal reinforcing bars flush with the concrete at the end of the column (Group 1) appeared to develop the full strength of their component parts and failed near the center without showing local weakness.

The columns with capitals developed high bending stresses and failed near the top.

The columns with dowels showed less strength than did those with continuous bars. Failure occurred at the ends of the bars and at the ends of the dowels.

The superiority of the columns with continuous bars flush with the surface of the concrete and having no capitals, together with the fact that they were the less expensive to make and test, resulted in the adoption of this type of column for the remainder of the investigation.

SERIES 2. EFFECT OF HOLDING THE LOAD FOR FOUR HOURS

12. *Purpose and Program*—The purpose of this series of tests was to ascertain whether the holding of load for a short period such as four hours, would have any effect on the strain and the ultimate strength of the column. There was the added purpose of comparing strengths of columns with three different grades of longitudinal reinforcement, structural, intermediate, and rail steel grades. Variations in percentage of reinforcement, and strength of concrete also were introduced as is indicated in Table 5.

The columns had an outside diameter of 8¼ in., a core diameter of 8 in., and an overall length of 60 in. The longitudinal bars were 60 in. long and the ends were milled, thus insuring direct bearing of both steel and concrete on the end plates. Three like columns were made for each group of plain concrete columns and two like for each group of reinforced concrete columns.

TABLE 5—PROGRAM OF TESTS FOR SERIES 2

Concrete Strength lb. per sq. in.			Reinforcement: Longitudinal Grade	Longitudinal Per cent	Spiral Grade	Spiral Per cent	Method of Loading		No. of Like Columns	Total No. of Columns
2000	3500	5000	...	0		0	Fast	Slow	3	18
2000	3500	5000	Interm. Struct. Rail	4	Interm.	1.2	Fast	Slow	2	36
	3500		Interm.	0 1.5 6	Interm.	1.2	Fast	Slow	2	12
	8000			0		0	Fast		3	3
	8000		Interm.	0 1.5 4 6	Interm.	1.2	Fast		2	8
									Total	77

13. *Proportioning and Strength of Concrete**—The concrete strength was determined on moist-cured 6 by 12-in. cylinders at an age of 56 days. The concrete designed for a cylinder strength of 2000 lb. per sq. in. was a 1:3.42:5.13 mix by weight of the dry materials. The assumption was made that the amount of water absorbed by the dry aggregates was 0.7 per cent of their weight. The cement content was 4.0 sacks per cu. yd. and the net water content was 39 gal. per cu. yd. of concrete or 9¾ gal. per sack of cement. The test results showed an average cylinder strength of 2280 lb. per sq. in. and an average slump of 2½ in. for all specimens included in this mix.

The concrete designed for a cylinder strength of 3500 lb. per sq. in. was of a 1:2.65:4.00 mix. The cement content was 5.02 sacks per cu. yd. and the net water content 39 gal. per cu. yd. of concrete or 7¾ gal. per sack of cement. The average strength was 3680 lb. per sq. in. and the average slump 3¾ in.

The concrete designed for a cylinder strength of 5000 lb. per sq. in. was a 1:1.98:2.96 mix. The cement content was 6.5 sacks per cu. yd. and the net water content 39 gal. per cu. yd. or 6 gal. per sack of cement. The average strength was 5360 lb. per sq. in., and the slump was 5½ in. It will be noted that the water content was 39 gal. per cu. yd. of concrete for all these mixes. A

*The cement and aggregates used in this series came from the same supplies as those used in the Preliminary Investigation reported in the JOURNAL for April, 1930.

placeable concrete was produced with this water content and only a small variation was noted in the consistency as measured by the slump.

In addition, 11 columns with concrete designed to have a compressive strength of 8000 lb. per sq. in. were made. The mix used for this concrete was 1:½:1 by weight. The cement content was 15.3 sacks per cu. yd. and the water content was 46 gal. per cu. yd., or 3 gal. per sack of cement. This mix was so dry that it could not be properly placed by ordinary methods. It was therefore worked into place by means of an electric vibrator furnished by the Electric Tamper and Equipment Company. The amount of vibration required was quite extreme, and even then small pockets were found in the interior of the cylinders when they were broken. No such pockets were found in the plain columns but no examination was made of the interior of the reinforced columns.

Fig. 19 shows the relation between the strength of the concrete and the water-cement ratio used in the mix. The water-cement strength relation for the plastic mixes was a smooth curve very similar to ordinary water-cement strength curves. The relation between cement content and strength is given by the straight, broken line. In other words, as long as the water content per cu. yd. of concrete was maintained constant with nearly constant consistency of the different mixes, the strength was directly proportional to the cement content.

The strength of the concrete designed for 8000 lb. per sq. in. proved to be only 7000 lb. per sq. in., and it falls below the water-cement strength curve defined by the other mixes. Neither does this strength fall upon the straight line determined by the other mixes when their strengths are plotted against the cement content. This 7000-lb. concrete contained 15.3 sacks of cement per cu. yd. and should have a strength of approximately 12,000 lb. per sq. in. in order to fall upon either of the strength curves shown in Fig. 19.

14. *Modulus of Elasticity of Concrete*—The modulus of elasticity was determined on one control cylinder from each column. The apparatus used consisted of two steel collars, 10 in. apart vertically, attached directly to the cylinders by means

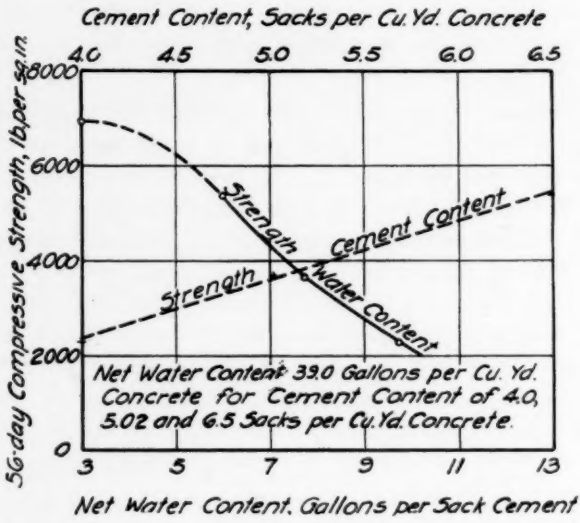


FIG. 19—RELATION BETWEEN STRENGTH AND WATER CONTENT AND BETWEEN STRENGTH AND CEMENT CONTENT FOR CONCRETE IN SERIES 2

of two set-screws 180 deg. apart. Opposite each of the set-screws a 1/10,000-in. Ames gage with its plunger pointing downward was clamped to the upper collar. A steel rod clamped to the lower collar opposite each set-screw extended upward and bore against the plunger of the Ames gage so that a direct reading of the total deformation in 10 in. was obtained. The results thus obtained were averaged for each type of concrete used and these averages are plotted in Fig. 20. The tangent modulus at a stress of 500 lb. per sq. in. was decided upon as the most reasonable value of the modulus of elasticity. In Fig. 21 the moduli of elasticity have been plotted against the strength. For all except the vibrated concrete, the increase in the modulus was very nearly proportional to the increase in strength. Even the value for the vibrated concrete falls not far above the projected straight line determined by the other values.

The results obtained indicate that the method used for deter-

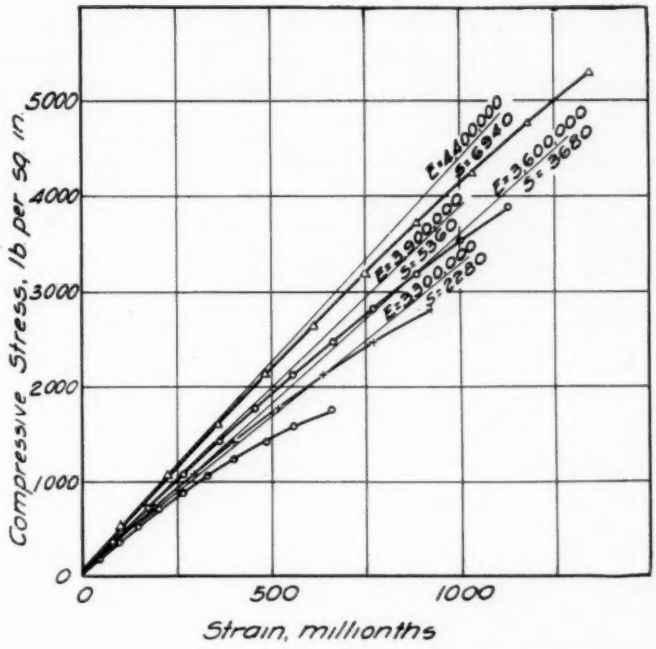


FIG. 20—AVERAGE STRESS-STRAIN DIAGRAM FOR CONTROL CYLINDERS OF SERIES 2

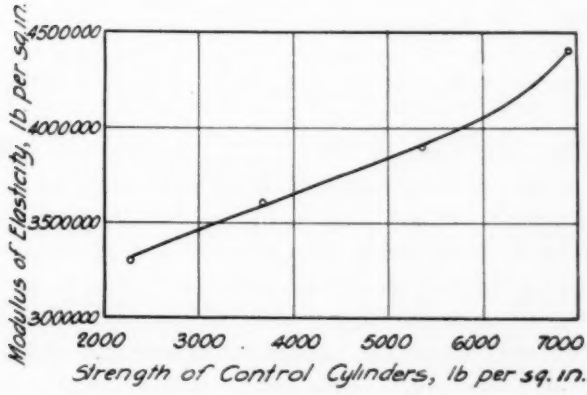


FIG. 21—RELATION BETWEEN MODULUS OF ELASTICITY AND STRENGTH OF CONTROL CYLINDERS FOR SERIES 2

TABLE 6—SUMMARY OF TESTS OF REINFORCEMENT

Nominal Size	No. of Coupons Tested	Yield-Point Stress lb. per sq. in.	Ultimate Strength lb. per sq. in.
Longitudinal Reinforcement—Intermediate Grade			
½-in. square	28	43,560	66,960
½-in. round	6	51,640	76,550
⅝-in. round	6	45,540	73,660
¾-in. round	6	45,470	69,220
		Structural Grade	
½-in. square	24	40,840	64,320
		Rail Grade	
½-in. square	21	65,320	102,680
Spiral Reinforcement—Intermediate Grade			
No. 5 (0.207 diam.)	65	Not Determined	82,760

mination of the modulus of elasticity was satisfactory. The simplicity of this method makes it very convenient for determination of modulus of elasticity where greater accuracy is not possible on account of time and expense involved.

15. *Reinforcement*—The longitudinal reinforcement consisted of intermediate, structural, and rail grades of steel bars. The intermediate and structural grades of bars were donated by the Concrete Steel Company of New York, and the rail steel by the Rail Steel Bar Assocation, Chicago. The bars were 22 ft. long when received. At the laboratory they were cut into four 5 ft. lengths and a 2-ft. test coupon. The four 5 ft. lengths were used in the same column. Four ½-in. diameter bars correspond to 1.5 per cent of longitudinal reinforcement, eight ½-in. square bars correspond to 4 per cent, and four ⅝-in. diameter plus four ¾-in. diameter bars correspond to 6 per cent of reinforcement. The coupons from each set of four bars were tested for yield-point stress and ultimate strength. The summary of these tests is given in Table 6. Both yield-point stress and the ultimate strength of the coupons (based upon nominal areas) varied considerably. The variation in weights of bars of the same size indicates that the variation in strength was probably due to the

variation of the cross-section from the nominal size of the bars, rather than differences in quality of the steel.

The yield point and ultimate strength for the structural grade were only slightly lower than those for the intermediate grade steel. The rail steel showed yield point and ultimate strength considerably higher than those for the intermediate grade. The marked influence of this difference will be pointed out later.

The lateral reinforcement consisted of wire spirals. The spiral wire was furnished by the American Steel and Wire Company, Chicago, and fabricated into spirals by the American System of Reinforcing, Chicago. The spirals had an outside diameter of 8 in., and an overall length of 59 in. Three extra turns were provided at each end of the spiral. A No. 5 wire having a 1.35-in. pitch gave 1.2 per cent lateral reinforcement. A test coupon was attached to each spiral. The average tensile strength on these coupons is given in Table 6. It will be noted that the ultimate strength of the No. 5 wire was considerably above that of the intermediate grade longitudinal reinforcement.

The reinforcement for each column was assembled into one unit before being placed in the mold. For the purpose of carrying the strain gage holes, ½-in. steel cubes were welded to four of the longitudinal bars at intervals of 10 in. along their length, thus forming a continuous row of five 10-in. gage lengths on each bar. The end cubes were placed five inches from the ends of the column.

The spirals had three spacers which were placed approximately 120 deg. apart. The longitudinal bars were fastened to the inside periphery of the spiral by means of wires.

16. *Placing and Curing of Concrete*—The mixing was done in a kettle-shaped mixer. The dry materials were mixed for two minutes, the water was then added, and the mixing was continued for four minutes more. One batch of concrete was sufficient for one column and its three control cylinders. For the plastic concrete continuous placing was used in the concreting of the columns. During the placing, the concrete was rodded continuously with a ½-in. steel rod and the mold was tapped with a

hammer in order to consolidate the mass. The concrete to be placed by the vibrator was placed in three equal layers. Each layer was vibrated for about 45 seconds.

About four hours after concreting, the columns were capped with a remixed neat cement paste. At the age of 24 hours the molds were removed, and a steel plug ¼ in. in diameter and ½ in. long was set in the concrete opposite each lug on the longitudinal bars. Neat cement paste was used to embed them in the concrete. The plugs in the concrete and the lugs on the bars were coated with "Cosmic anti-rust preparation," and the columns were placed in the moist room. At the age of seven days the columns were removed from the moist room, holes were drilled in the plugs and lugs, strain gage observation were taken, and the columns were replaced in the moist room to remain until an age of 56 days was reached.

17. *The Testing*—At the age of 56 days the columns were removed from the moist room and placed in the 800,000-lb. screw-power testing machine. The testing crew consisted of two men, an observer and a recorder. Strain gage readings were taken on the entire 40 gage lengths before applying any load and again at each increment of load. About 10 increments were used. The time required for a full set of observations was about seven minutes. With the "fast" loading the weighing beam was re-balanced after completion of each set of readings and a new increment of load was applied immediately.

With the "slow" loading the procedure was similar to that for fast loading up to about 70 per cent of the maximum load. From that time on four hours elapsed between successive increments of load, and readings were taken immediately after completion of one increment and again just before starting of the next one. During the interval between increments the load was kept within about 2000 lb. of that intended. This was done by "bringing up the load" whenever it had fallen off appreciably. The frequency with which the machine had to be operated to maintain the load varied from a few minutes to more than half an hour within any given period between increments. Near the maximum load the frequency with which it was necessary to bring up the load was much greater than for the lower loads. To prevent drying out

during the four-hour period in which the load was held, the column was enclosed in a canvas jacket which was kept moist by sprinkling. All the columns were photographed after completion of the test. A measure of the reliability of the experimental work of this investigation is found in the close agreement of measured strains (1) between the steel and the concrete of the same column, (2) between companion columns of any given group, (3) between

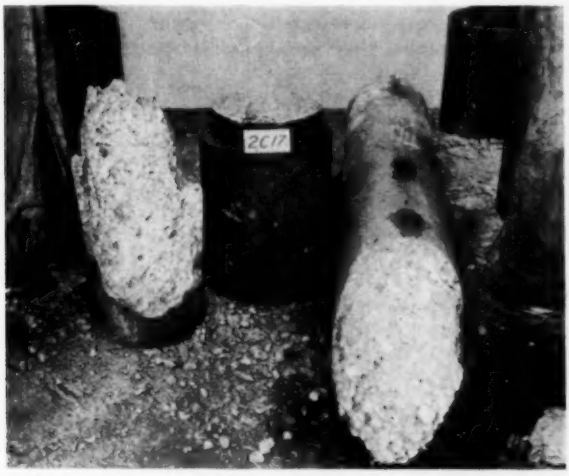


FIG. 22—A PLAIN CONCRETE COLUMN AFTER FAILURE

two groups of the same kind, one of which was used for fast loading and the other for slow loading. This agreement indicates not only good strain gage work but uniformity of making, capping, and curing of the test specimens.

18. *Plain Columns*—The ultimate strengths of the plain columns and of their control cylinders are given in Table 7, also the strengths of the columns stated as a percentage of the strength of the control cylinders. The average strength of the plain columns was approximately 85 per cent of the strength of the cylinders and the uniformity of both the cylinders and the columns was good. The plane of failure generally made an angle of about 30 deg. with the axis of the column as illustrated in Fig. 22. Failure generally occurred within the upper half of

TABLE 7—STRENGTH OF PLAIN CONCRETE COLUMNS UNDER FAST LOADING AND SLOW LOADING

Strengths are given in pounds per square inch

Method of Loading Columns	Strength of Control Cylinders				Designed Strength
	For Column No.			Average	
	1	2	3		
Fast	2120	2000	2210	2110	2000
Slow	2250	2220	2080	2180	2000
Fast	3000	3600	3420	3340	3500
Slow	3220	3450	3450	3370	3500
Fast	5050	5260	5280	5200	5000
Slow	5130	5120	5480	5240	5000
Fast	7470	7450	7060	7330	8000
Method of Loading Columns	**Strength of Columns**				**Per cent Cylinder Strength**
Fast	1820	1850	1680	1780	84.5
Slow	1680	1680	1680	1680	77.8
Fast	2790	3010	3050	2950	88.5
Slow	2800	2520	2800	2710	80.5
Fast	4290	4470	4130	4300	82.8
Slow	4770*	4770*	....†	4770*	91.1*
Fast	6680	5930	7070	6560	89.5
				Av.	85.0

*Accidentally run beyond the last 10 per cent increment.
†Tipped over and broken before the testing.

the column, a fact which is consistent with the observation in Series 1, that the eccentricity of loading was largest near the top.

The columns of 8000-lb. concrete failed explosively. Those with 5000-lb. concrete failed less abruptly but almost without warning. The columns of 3500-lb. concrete failed gradually and in some cases it was possible to get them out of the machine whole. The columns with 2000-lb. concrete settled down gradually under the load and were removed from the machine in one piece.

The stress-strain curves for the cylinders and the columns tested with fast loading are given in Fig. 23, 24, 25, and 26, for

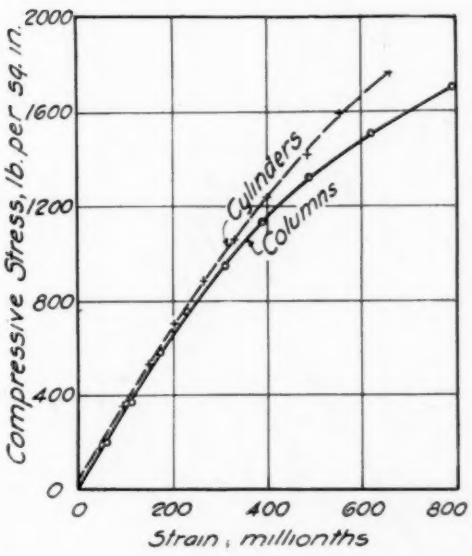


FIG. 23—STRESS-STRAIN DIAGRAM FOR PLAIN COLUMNS AND FOR CONTROL CYLINDERS; DESIGNED CONCRETE STRENGTH 2000 LB. PER SQ. IN.

the four strengths of concrete. Each curve is the average for all the specimens of its kind. The solid lines represent the columns and the dotted lines represent the cylinders. The outstanding features of these figures are the close agreement between the curves for the cylinders and those for the columns within the lower half, and the consistently greater deformation in the columns within the upper half of the curves. Generally the curve for the cylinders lies slightly above that for the columns even at low loads, indicating apparently, that there was some slight delay in getting the strain measuring apparatus used on the cylinders into action.

The modulus of elasticity given by the slope of the stress-strain curves for the columns for a stress of 500 lb. per sq. in. is in close

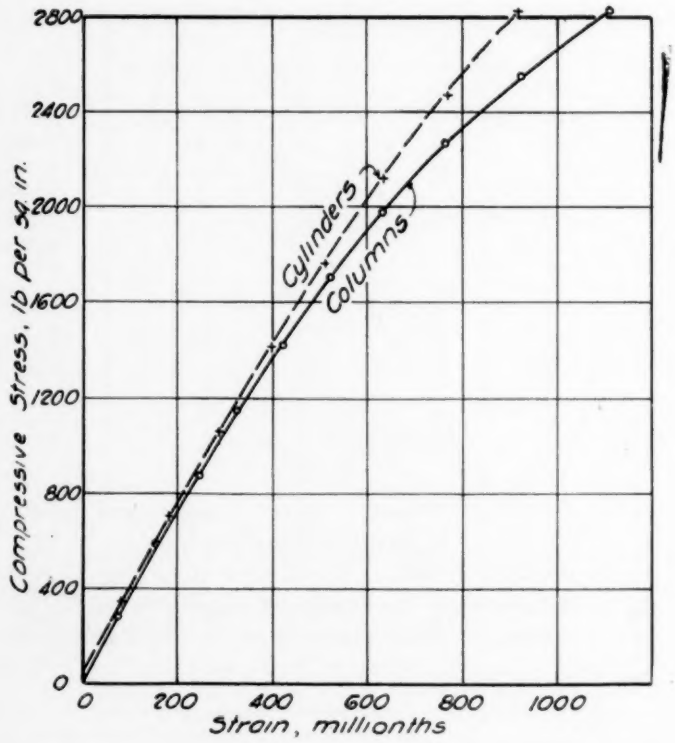


FIG. 24—STRESS-STRAIN DIAGRAM FOR PLAIN COLUMNS AND FOR CONTROL CYLINDERS; DESIGNED CONCRETE STRENGTH 3500 LB. PER SQ. IN.

agreement with that found for the cylinders.

Fig. 27 shows the stress-strain curves for fast and slow loading of columns designed for strengths of 2000, 3500, and 5000 lb. per sq. in. The horizontal portions of the curves represent the deformation which occurred during the four hours under practically constant load. The outstanding feature is the close agreement of those portions of the load strain curves in which the load was applied rapidly.

The horizontal distance between the upper parts of the two curves for the same concrete strength represents the portion of

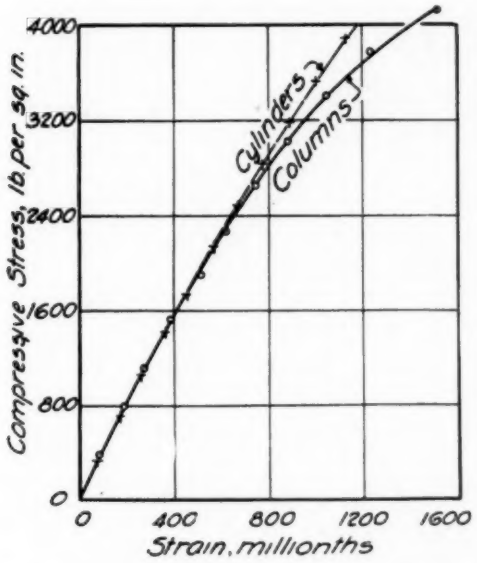


FIG. 25—STRESS-STRAIN DIAGRAM FOR PLAIN COLUMNS AND FOR CONTROL CYLINDERS; DESIGNED CONCRETE STRENGTH 5000 LB. PER SQ. IN

the total deformation which is due to the holding of the load for four hours.

19. *Relation Between Strains and Percentage of Longitudinal Reinforcement*—Figs. 28 and 29 show the stress-strain relations for columns of concrete designed for strengths of 8000 and 3500 lb. per sq. in. respectively. The spiral reinforcement amounted to 1.2 per cent for all the columns, and the longitudinal reinforcement varied from 0 to 6 per cent as indicated. The stress $\frac{P}{A}$ plotted in these diagrams is the load on the column divided by the total area within the outer circumference of the spiral, that is, by 50.3 sq. in., the sum of the sectional areas of the circumscribed concrete and the longitudinal reinforcement. The portion of this average stress that was considered to be carried by the longitudinal reinforcement is indicated by the broken straight lines in the lower part of the diagram. In constructing

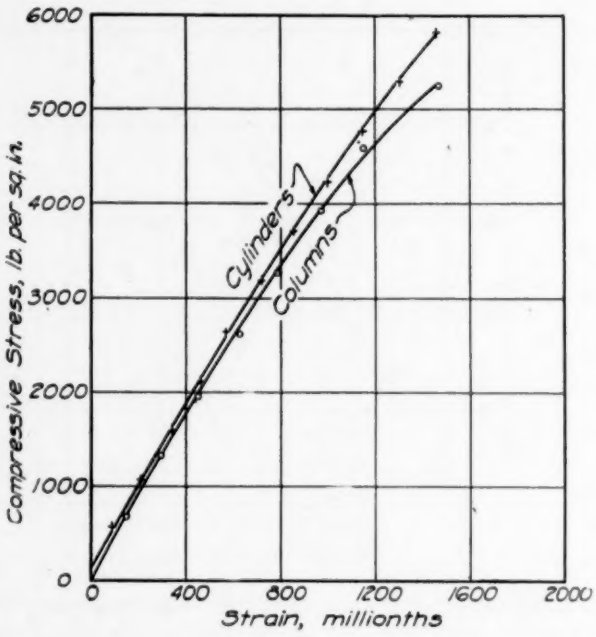


FIG. 26—STRESS-STRAIN DIAGRAM FOR PLAIN COLUMNS AND FOR CONTROL CYLINDERS; DESIGNED CONCRETE STRENGTH 8000 LB. PER SQ. IN.

these lower lines the modulus of elasticity of the steel was taken at 30,000,000 lb. per sq. in. and the stresses were determined from the measured strains. The break in each line comes at the strain corresponding to the yield-point stress of the bars of the size used to secure the percentage of reinforcement indicated.

The stress-strain curves for the control cylinders made with the columns having only spiral reinforcement, have been plotted in Figs. 28 and 29 also. In each figure the curve for the cylinder is a dotted line and its agreement with the curve for the corresponding column is very good. Figs. 28 and 29 indicate much less departure of the strain in the spiraled columns from that in their control cylinders than is shown in Fig. 23 and 26 for the plain columns. The indication is that the presence of the spiral had a slight effect in reducing the longitudinal strain in the

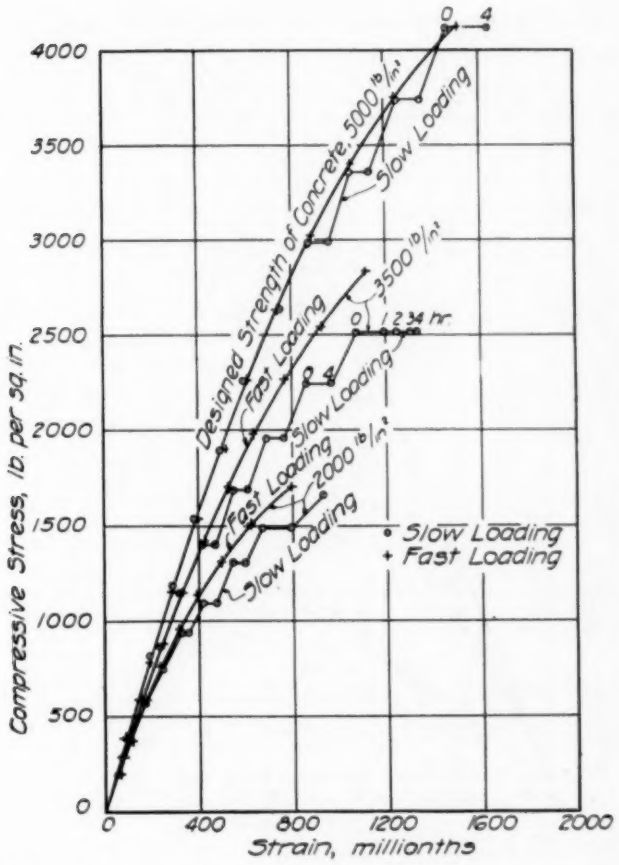


FIG. 27—STRESS-STRAIN DIAGRAM FOR PLAIN CONCRETE COLUMNS, FAST AND SLOW LOADING

columns for stresses above about one-half the cylinder strength, but no effect for lower stresses.

For the concrete designed for a compressive strength of 8000 lb. per sq. in., Fig. 28, the vertical distances between the stress-strain curves for the columns with various percentages of longitudinal reinforcement are very nearly the same as the vertical distances between the corresponding straight lines representing

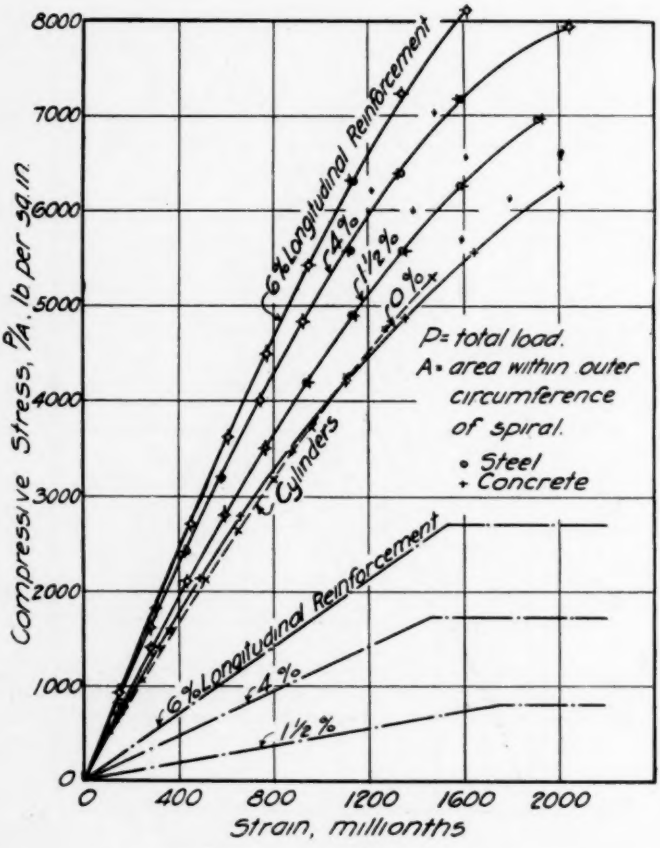


FIG. 28—STRESS-STRAIN DIAGRAM FOR COLUMNS WITH VARYING PERCENTAGES OF LONGITUDINAL REINFORCEMENT; DESIGNED CONCRETE STRENGTH 8000 LB. PER SQ. IN.; 1.2 PER CENT SPIRAL REINFORCEMENT

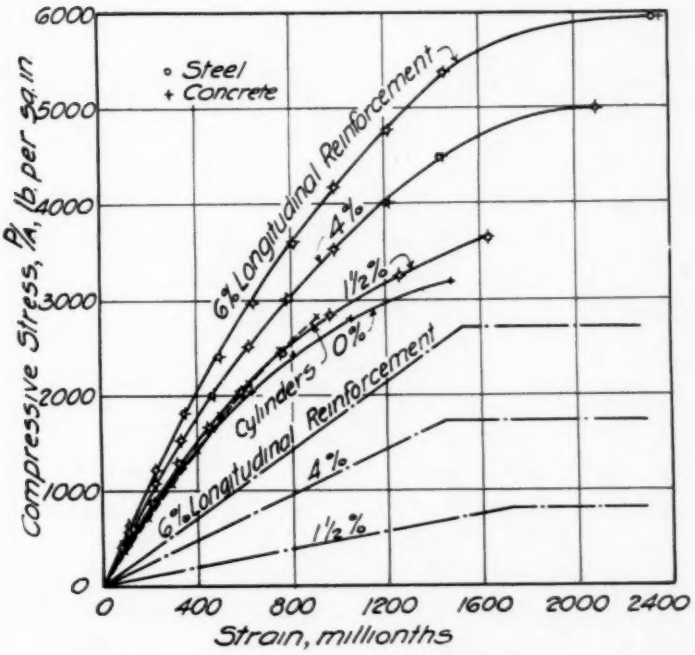


FIG. 29—STRESS-STRAIN DIAGRAM FOR COLUMNS WITH VARYING PERCENTAGES OF LONGITUDINAL REINFORCEMENT; DESIGNED CONCRETE STRENGTH 3500 LB. PER SQ. IN.; 1.2 PER CENT SPIRAL REINFORCEMENT, FAST LOADING

the stresses carried by the reinforcement. That is, if the ordinates to the straight lines (the stresses carried by the longitudinal reinforcement) were subtracted from the ordinates to the stress-strain curves for the columns, the resulting curves would practically coincide with the curve for the column having no longitudinal reinforcement. This indicates that the presence of longitudinal reinforcement had no perceptible effect on the elastic behavior of the concrete designed for a strength of 8000 lb. per sq. in.

With the 3500-lb. concrete shown in Fig. 29 for fast loading and Fig. 30 for slow loading, the case was different. The vertical distances between the stress-strain curves for 0 and 1½ per cent

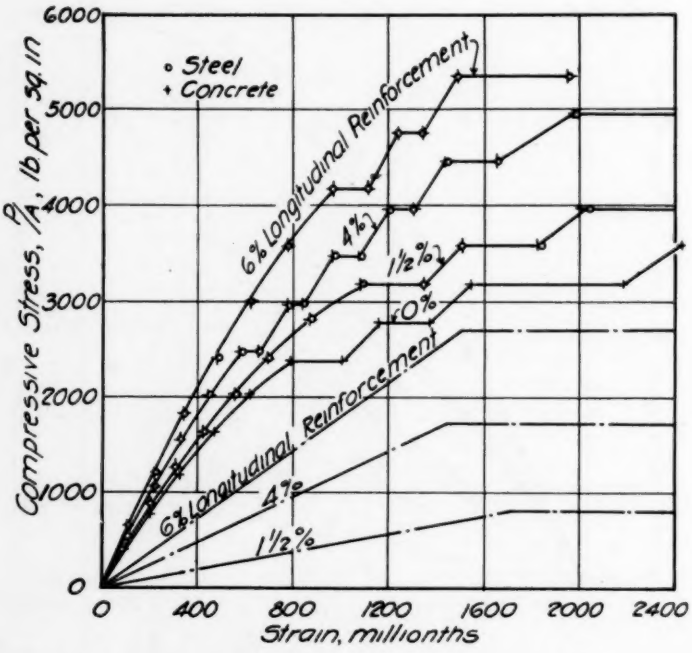


FIG. 30—STRESS-STRAIN DIAGRAM FOR COLUMNS WITH VARYING PERCENTAGES OF LONGITUDINAL REINFORCEMENT; DESIGNED CONCRETE STRENGTH 3500 LB. PER SQ. IN.; 1.2 PER CENT SPIRAL REINFORCEMENT, SLOW LOADING

reinforcement were considerably less than the ordinates of the line representing the stresses carried by 1½ per cent of reinforcement. With this exception, the conditions for the 3500-lb. concrete were about the same as those for the 8000-lb. concrete with the result that if the stresses carried by the reinforcement were subtracted from the total average stresses carried by the corresponding columns the resulting curves for the stresses carried by the concrete for the reinforced columns would practically coincide with each other, but would lie below the curve for the columns without longitudinal reinforcement. The behavior is as though the introduction of the longitudinal reinforcement reduced the effective sectional area of the concrete by a constant amount in

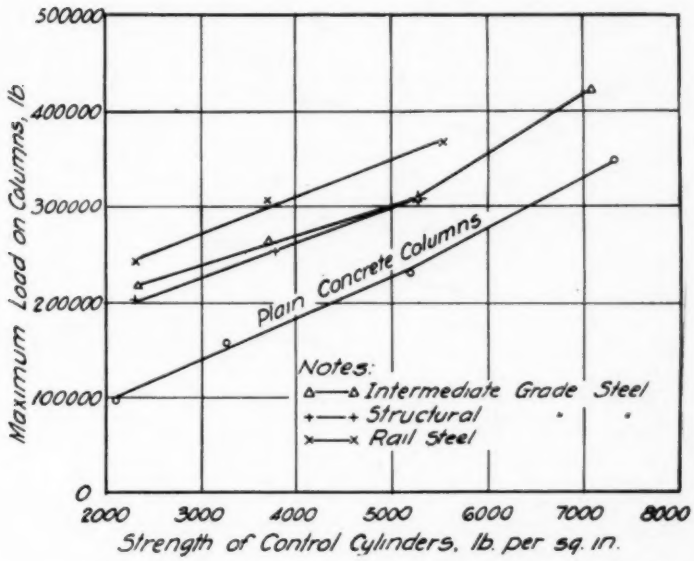


FIG. 31—COMPARISON OF PLAIN CONCRETE COLUMNS WITH COLUMNS HAVING 4 PER CENT LONGITUDINAL REINFORCEMENT OF THREE DIFFERENT GRADES, 1.2 PER CENT SPIRAL REINFORCEMENT OF INTERMEDIATE GRADE

excess of the area of the reinforcement used. The only explanation of the difference in behavior between the 8000 and 3500-lb. concrete which seems reasonable is based upon a possible difference in compactness due to the difference in the methods of placing. The 8000-lb. concrete was vibrated during the placing, and the 3500-lb. concrete was not.

20. *Relation Between Strength of Column and Strength of Concrete*—Fig. 31 shows, as ordinates, the total loads carried by the columns and as abscissas, the strengths of their control cylinders. The lower curve represents the plain columns and the upper curves represent columns having 4 per cent longitudinal and 1.2 per cent spiral reinforcement. The spiral reinforcement was all of intermediate grade; the longitudinal included structural,

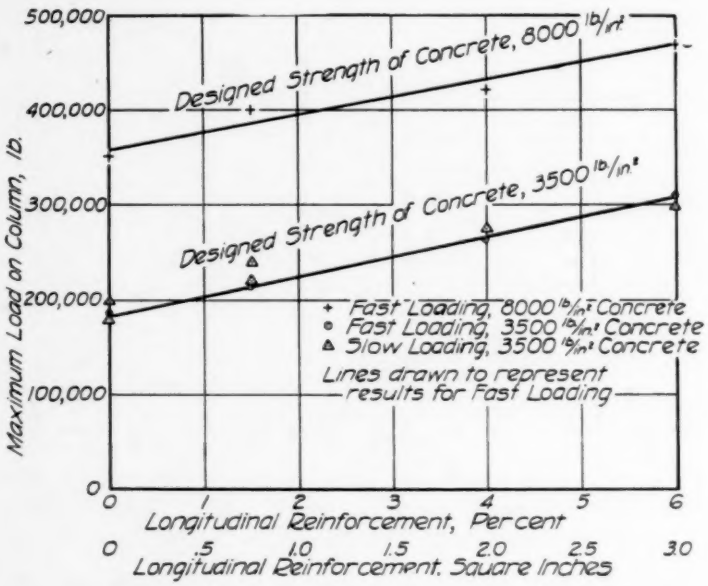

500,000
400,000
300,000
200,000
100,000
0
Maximum Load on Column, lb.
Designed Strength of Concrete, 8000 lb/in²
Designed Strength of Concrete, 3500 lb/in²
+ Fast Loading, 8000 lb/in² Concrete
○ Fast Loading, 3500 lb/in² Concrete
△ Slow Loading, 3500 lb/in² Concrete
Lines drawn to represent results for Fast Loading
0 1 2 3 4 5 6
Longitudinal Reinforcement, Per cent
0 .5 1.0 1.5 2.0 2.5 3.0
Longitudinal Reinforcement, Square Inches

FIG. 32—EFFECT OF VARIATION IN PERCENTAGE OF LONGITUDINAL REINFOPCEMENT ON THE STRENGTH OF THE COLUMNS, 1.2 PER CENT SPIPAL REINFORCEMENT

intermediate, and rail grades of steel. The strengths of the columns increased quite consistently with the increase in strength of the control cylinders. The rate of increase in strength of the reinforced columns seems to have been nearly equal to that of the plain columns when net core areas* of concrete in the reinforced columns are considered. That is, the increase in the strength of the concrete was just as effective in a reinforced column as it was in a plain column. It is shown in a previous section that for a plain column the strength was 85 per cent of its cylinder strength regardless of the strength of the concrete. Therefore the increase in strength of the reinforced columns would be equal to the net area of its concrete times 85 per cent of the increase in strength of its control cylinders, regardless of the grade of longitudinal reinforcement.

*By the term "net core area" of the concrete is meant the area within the outer circumference of the spiral minus the sectional area of the longitudinal reinforcement.

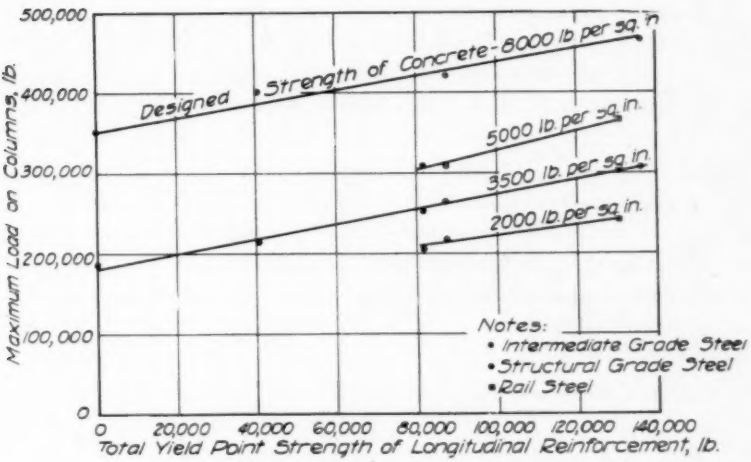


FIG. 33—RELATION BETWEEN TOTAL YIELD-POINT STRENGTH OF LONGITUDINAL REINFORCEMENT AND STRENGTH OF COLUMN, 1.2 PER CENT SPIRAL REINFORCEMENT

In Fig. 32 the maximum loads carried by reinforced columns have been plotted as ordinates and the percentage of longitudinal reinforcement as abscissas. The percentage of spiral was 1.2 for all columns. The straight line representing the relation between the strength of the column and the percentage of reinforcement for a concrete strength of 3500 lb. per sq. in. is parallel to the line representing the same relation for the 8000-lb. concrete. That is, the strength added to a reinforced column due to a given increase in the concrete strength was the same for all the percentages of longitudinal reinforcement used.

Fig. 33 shows the relation between the strength of the columns and the total yield-point strength of the longitudinal reinforcement. This figure is a combination of the previous two figures and shows that the increase in strength of columns due to increased strength of the concrete was practically independent of the yield point strength of the reinforcement used in the columns.

The effect of the strength of the concrete upon the deformation of columns having 4 per cent longitudinal and 1.2 per cent spiral

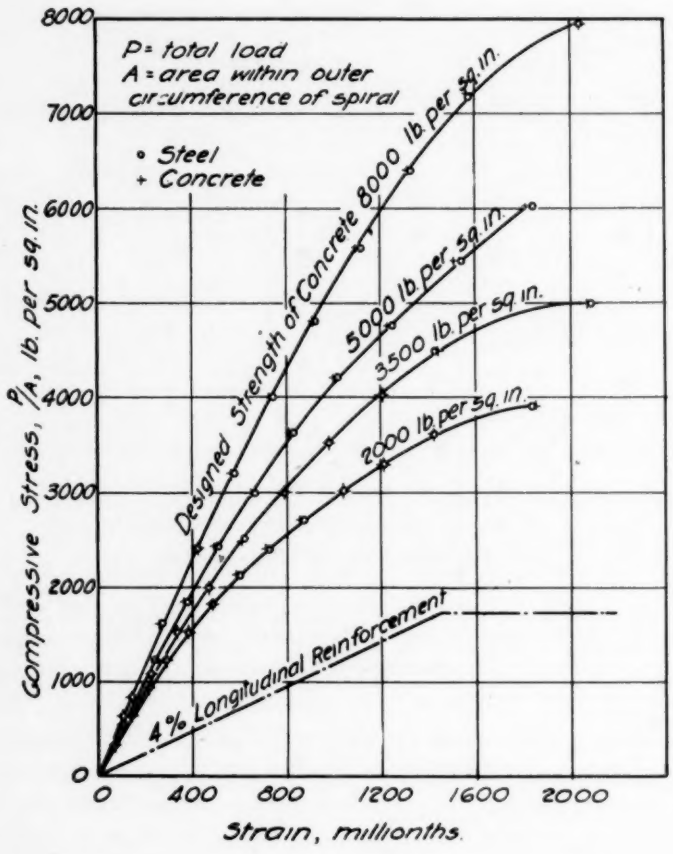


FIG. 34—STRESS-STRAIN DIAGRAM FOR COLUMNS WITH DIFFERENT CONCRETE STRENGTHS, REINFORCEMENT 4 PER CENT LONGITUDINAL AND 1.2 PER CENT SPIRAL

reinforcement is illustrated in Fig. 34. The strains for a given stress decreased quite regularly with the increase in strength of the concrete.

21. *Relation Between Yield-Point Strength of Longitudinal Reinforcement and Strength of Column*—The strength added by the longitudinal reinforcement in the columns may be varied by changing either the percentage or the grade of reinforcing steel.

The effect of various grades of steel upon the strength of columns of a given percentage of reinforcement and strength of concrete is illustrated in Fig. 31. Intermediate, structural, and rail grades were included in these tests, and the column strengths obtained with these grades of longitudinal reinforcement are shown in the three upper lines. The yield-point strength of the reinforcement is given in Table 6. Structural and intermediate grades had nearly equal yield-point stresses (40,840 and 43,560 lb. per sq. in.), but the rail grade had a much higher yield-point stress (65,320 lb. per sq. in.). The lines representing the strengths of the columns having structural and intermediate grades of reinforcement are close together, while the line representing the strength for the columns with rail grade reinforcement lies considerably higher than the lines for the other grades.

In Fig. 33 the strengths of the columns have been plotted as ordinates and the total yield-point strength of the longitudinal steel as abscissas. Straight lines are obtained for each strength of concrete. The increase in yield-point strength was due in some cases to change in grade of steel, and in other cases to increased percentage of reinforcement. Since for a given strength of concrete, all points group themselves along the same line, the strength of the column seemed to be independent of whether the change in yield-point strength was affected by varying the percentage of a given grade of reinforcement or by using different grades of reinforcement. The lines for different strengths of concrete are practically parallel, indicating that the increase in strength due to the higher yield-point strength of the reinforcement was independent of the strength of the concrete. The slope of these lines represents the strength added to the columns for each unit of increase in strength of the reinforcement. The slope is, therefore, a measure of the effectiveness of the longitudinal reinforcement in adding strength to a column of a given concrete strength. The average slope of the curves in Fig. 33 indicate that the reinforcement added 97 per cent of its yield-point strength to the strength of the concrete of the core. The yield-point stress of the steel was determined by the "drop of beam," a method which tends to give values slightly too high. The effectiveness of the longitudinal reinforcement may therefore be

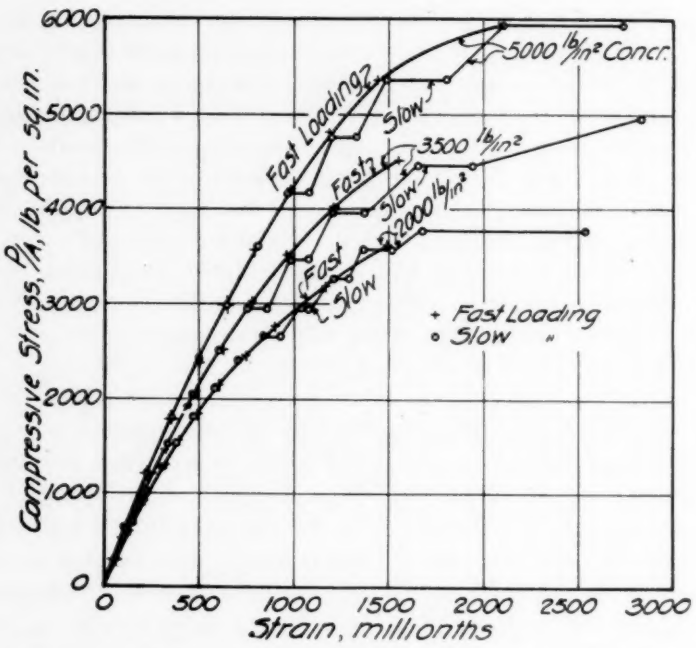


FIG. 35—STRESS-STRAIN DIAGRAM FOR COLUMNS TESTED UNDER FAST LOADING AND SLOW LOADING; 4 PER CENT LONGITUDINAL REINFORCEMENT OF STRUCTURAL GRADE, 1.2 PER CENT SPIRAL REINFORCEMENT

more than 97 per cent of its yield-point strength. In the further discussion in this report the longitudinal reinforcement has been assumed to add its full yield-point strength to the strength of the column. That is, it is considered 100 per cent effective.

22. *Effect of Rate of Loading on Deformation and Strength of Columns*—In Fig. 31 it is shown that the reinforced columns continued to deform during the 4-hour loading intervals. Up to the first load which was maintained for four hours, the deformations of the two sets of columns (designed for "fast" and "slow" loading) were practically equal in all cases, indicating good uniformity in the two sets of columns. The deformation during the first 4-hour interval was generally quite large. The deformation during the succeeding 4-hour intervals was not much different

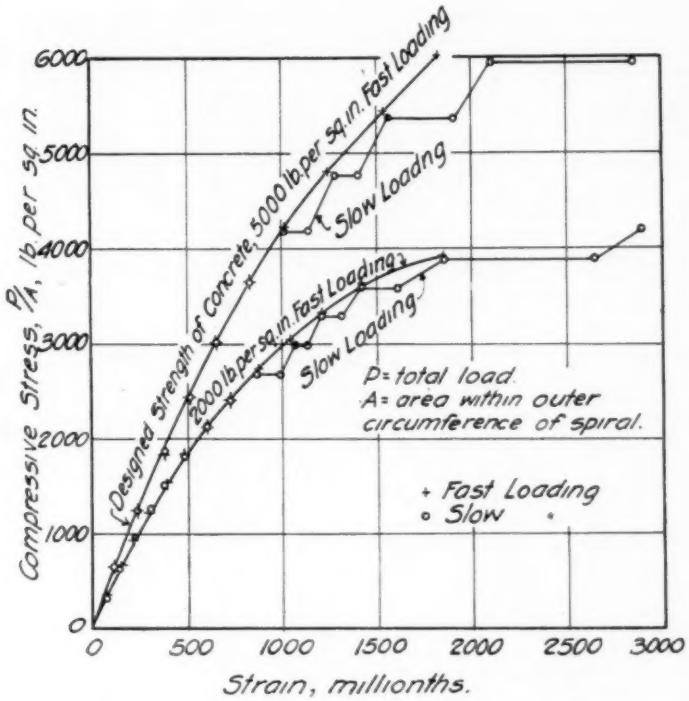


FIG. 36—STRESS-STRAIN DIAGRAM FOR COLUMNS TESTED UNDER FAST LOADING AND SLOW LOADING; 4 PER CENT LONGITUDINAL REINFORCEMENT OF INTEFMEDIATE GRADE, 1.2 PER CENT SPIRAL REINFORCEMENT

from the first one except near the maximum load, when the deformations became several times as large as during the previous intervals. During the application of an increment of load, the rate of deformation was so much lower than that for fast loading that the stress-strain curve was brought very nearly back to the curve for the fast loading.

Figs. 35, 36, and 37 show the deformation for fast and slow loading on columns reinforced with structural, intermediate, and rail grade longitudinal reinforcement. The grade of reinforcement had no effect upon the deformation up to a load near the yield-point strength of the steel. The agreement between de-

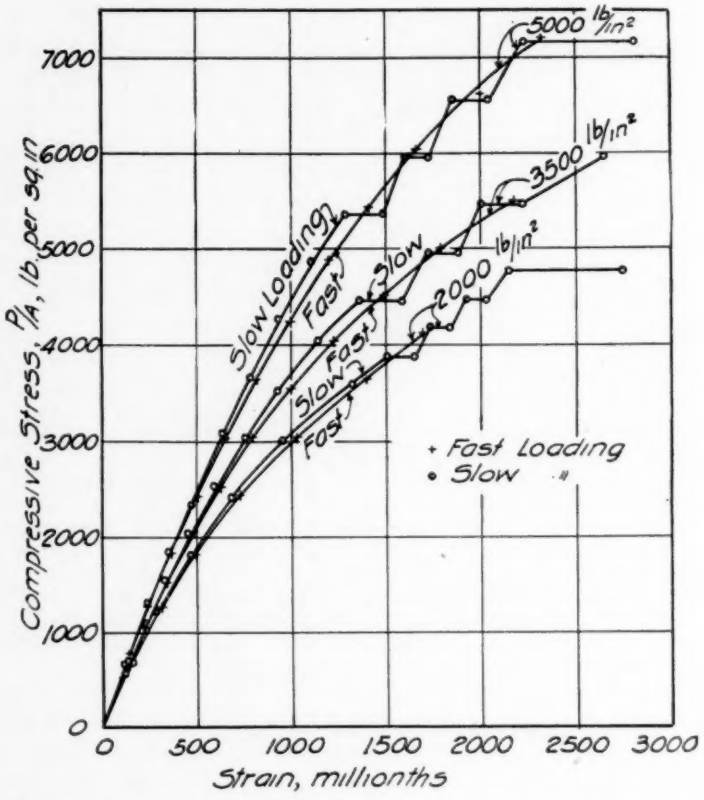


FIG. 37—STRESS-STRAIN DIAGRAM FOR COLUMNS TESTED UNDER FAST LOADING AND SLOW LOADING; 4 PER CENT LONGITUDINAL REINFORCEMENT OF RAIL GRADE, 1.2 PER CENT SPIRAL REINFORCEMENT

formations at low loads for the two methods of testing is evident. In Fig. 37, showing the deformation of the columns having rail steel, the agreement between deformations found for fast and slow loadings was not so good as in the other cases.

The strengths of the columns tested with fast loading and slow loading are given in Tables 7 to 10. Table 7 gives the results for plain columns, and Tables 8, 9, and 10 for reinforced columns. The plain columns tested with slow loading generally developed less strength than did the companion columns with fast loading.

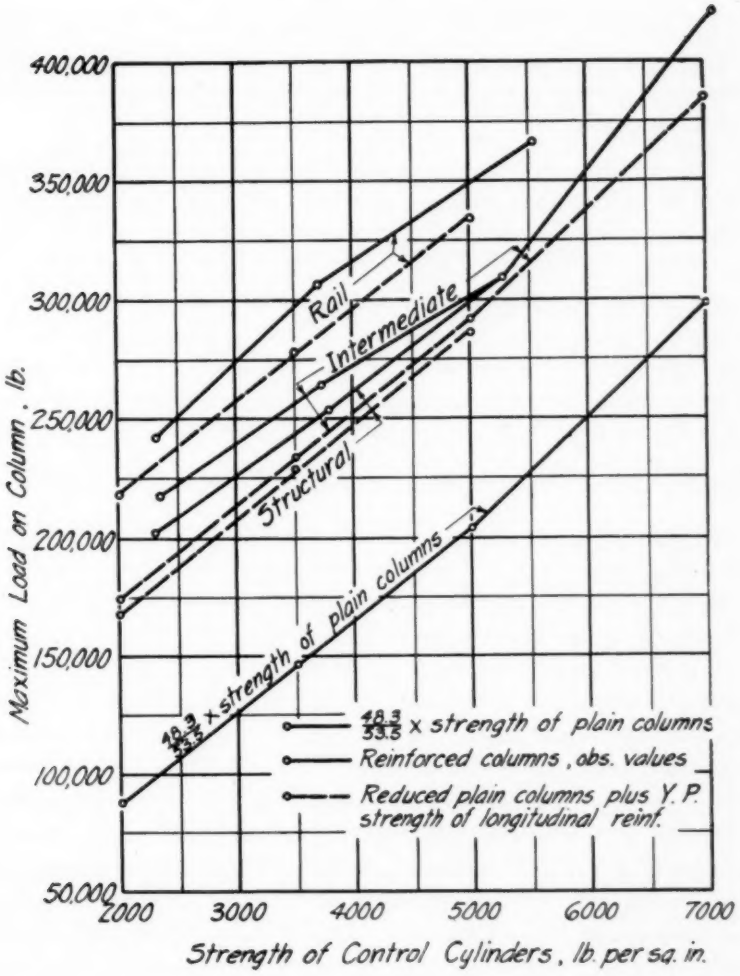


FIG. 38—DIVISION OF LOAD BETWEEN CONCRETE AND REINFORCEMENT

The difference, however, was not very marked. For the reinforced columns the strength obtained with slow loading was very nearly equal to, but generally larger, than the strength obtained with fast loading.

23. *Division of Load Between Concrete and Reinforcement*—Fig. 38 is designed to show the relation between the strengths of the reinforced columns and the sum of the strengths of the concrete and the reinforcement used. All the reinforced columns used in this figure had 4 per cent longitudinal and 1.2 per cent spiral reinforcement. The sectional area of the concrete within the outer circumference of the spiral, that is, the core area minus the sectional area of the longitudinal bars was 48.3 sq. in., and the sectional area of the plain columns was 53.5 sq. in. The plotted values in the lower curve are $\frac{48.3}{53.5}$ times the strength of the plain columns, and represent the strength of the concrete in the reinforced columns. To these strengths have been added the total yield-point strengths of the longitudinal reinforcement. The strengths thus obtained for columns reinforced with structural, intermediate, and rail grades of reinforcement are shown as three distinct dotted lines. In addition, the strengths of the columns so reinforced but having in addition 1.2 per cent spiral reinforcement are shown as solid lines. Based upon the assumption that the longitudinal reinforcement added its full yield-point strength to the column (as indicated in section 21), the vertical distance between the solid and the dotted lines for each grade of steel represents the strength added by 1.2 per cent of spiral reinforcement. The solid line and the dotted line are not far from parallel, indicating that nearly a constant strength was added to the column by the spiral regardless of the strength of the concrete. The average total added strengths so determined were 16,700, 19,000, and 16,000 lb. for longitudinal reinforcement of structural, intermediate, and rail grades of steel respectively. Apparently the strength added by the spiral was not affected by variations in the yield-point strength of the longitudinal reinforcement. The average of the added strengths for the three grades was 17,200 lb. for 1.2 per cent of spiral, or 14,300 lb. for 1

TABLE 8—EFFECT OF GRADE OF LONGITUDINAL REINFORCEMENT ON STRENGTH OF COLUMNS

Reinforcement in all columns: 4 per cent longitudinal and 1.2 per cent spiral.

Concrete Strength				Load on Column at Failure			Method of Loading
Designed	Obtained			1	2	Av.	
	1	2	Av.				
Intermediate Grade							
2000	2360	2340	2350	221,000	214,200	217,600	Fast
2000	2360	2400	2380	*1.5 hr. 210,000	3.5 hr. 210,000	2.5 hr. 210,000	Slow
3500	3790	3660	3730	260,500	268,000	264,250	Fast
3500	3810	3980	3900	*15 min. 275,000	15 min. 275,000	15 min. 275,000	Slow
5000	5150	5410	5280	321,000	297,500	309,250	Fast
5000	5160	5670	5420	*0 330,000	5 min. 300,000	—	Slow
Structural Grade							
2000	2250	2360	2310	201,500	202,700	202,100	Fast
2000	2320	2370	2350	*5 min. 210,000	40 min. 210,000	25 min. 210,000	Slow
3500	3750	3810	3780	252,600	253,000	252,800	Fast
3500	3830	3800	3820	*25 min. 225,000	30 min. 250,000	—	Slow
5000	5180	5350	5270	311,500	308,000	309,750	Fast
5000	5270	5480	5380	*0 330,000	45 min. 300,000	—	Slow
Rail Grade							
2000	2380	2230	2310	239,300	244,700	242,000	Fast
2000	2260	2520	2390	max. 260,200	*7 min. 255,000	—	Slow
3500	3650	3740	3700	307,000	304,700	305,850	Fast
3500	3800	3780	3790	*25 min. 300,000	0 325,000	—	Slow
5000	5460	5620	5540	355,000	378,000	366,750	Fast
5000	5530	5800	5670	*0 390,000	0 390,000	0 min. 390,000	Slow

*Time from application of maximum load until failure.

TABLE 9—EFFECT OF VARIATION IN PERCENTAGE OF LONGITUDINAL REINFORCEMENT ON STRENGTH OF COLUMNS

Fast Loading

Per cent Reinforcement		Cylinder Strength	Maximum Load at Failure	Assumed Load Distribution		Nominal Area of Long. Rein.
Longitudinal	Spiral			Concrete 85% of Cyl. Strength	Attributed to Presence of Reinforcement	
		lb. per sq. in.	lb.	lb.	lb.	sq. in.
Designed Cylinder Strength, 8000 lb. per sq. in.						
0	0	7330	348,300	348,300	0	0
0	1.2	6590	350,400	282,000	68,400	0.
1½	1.2	6600	400,150	278,000	122,150	0.785
4	1.2	7070	421,500	290,000	131,500	2.00
6	1.2	6950	470,000	280,000	190,000	2.99
Designed Cylinder Strength, 3500 lb. per sq. in.						
0	0	3340	157,900	157,900	0	0
0	1.2	3760	185,700	160,700	25,000	0
1½	1.2	3410	213,900	143,500	70,400	0.785
4	1.2	3730	264,250	153,000	111,250	2.00
6	1.2	3770	308,450	151,450	157,000	2.99

TABLE 10—EFFECT OF VARIATION IN PERCENTAGE OF LONGITUDINAL REINFORCEMENT ON STRENGTH OF COLUMNS

Designed strength of concrete—3500 lb. per sq. in.
Spiral reinforcement—1.2 per cent

Per cent Longitudinal Reinforcement	Concrete Strength Obtained lb. per sq. in.			Load on Columns lb.			Method of Loading
	1	2	Av.	1	2	Av.	
0.0	3790	3730	3760	186,900	184,500	185,700	Fast
0.0	3920	3810	3870	35 min. 200,000	85 min. 180,000	—	Slow
1.5	3710	3100	3410	218,500	209,300	213,900	Fast
1.5	3920	3800	3860	15 min. 220,000	0 240,000	—	Slow
4.0	3790	3660	3730	260,500	268,000	264,250	Fast
4.0	3810	3980	3900	15 min. 275,000	15 min. 275,000	15 min. 275,000	Slow
6.0	3850	3680	3770	305,000	311,900	308,450	Fast
6.0	3690	3790	3740	15 min. 300,000	5 min. 300,000	10 min. 300,000	Slow

per cent. This is 70 per cent of the average strength added by 1 per cent of longitudinal reinforcement of intermediate grade as shown in Fig. 31.

The indication of the foregoing paragraphs is that the strength of the columns having longitudinal and spiral reinforcement was made up of (1) 85 per cent of the cylinder strength times the net area of the concrete within the outer circumference of the spiral, (2) the yield-point stress of the longitudinal bars times their area, and (3) the strength added by the spiral, which amounted to about 70 per cent of the strength added by an equal percentage of longitudinal reinforcement of intermediate grade steel.

24. *Summary of Results*—With the water content of the concrete constant at 39 gal. per cu. yd. the strength of the concrete increased in direct proportion to the increase in the cement content.

For all except the vibrated concrete, the increase in the modulus of elasticity was very nearly proportional to the increase in the compressive strength.

The strains measured on the steel agreed very well with those measured on the concrete for all columns, and the agreement between strains measured on columns of the same kind was good.

The average strength of the plain columns was approximately 85 per cent of the strength of the cylinders and the uniformity of both the cylinders and the columns was good.

The stress-strain curves for the plain columns practically coincided with those for the corresponding control cylinders up to about one-half the maximum load. For higher loads the strains in the columns were somewhat greater than those in the cylinders.

For the 8000-lb. concrete the load at any strain for columns with different percentages of longitudinal reinforcement was equal to the load carried by a column with no longitudinal reinforcement plus the load carried by longitudinal reinforcement having a modulus of elasticity of 30,000,000 lb. per sq. in. With the 3500-lb. concrete the conditions were the same as for the 8000-lb. concrete except that for any given strain the loads for the columns with all the different percentages of reinforcement were a constant amount less than the sum of the load for the column

without longitudinal reinforcement and the load found to be carried by the reinforcement.

For any two columns reinforced in the same manner, the difference in total strength was approximately equal to the difference in strengths of the concrete in the core as determined by the tests of the plain columns.

The increase in total strength due to the use of longitudinal reinforcement was approximately equal to the yield-point stress of the reinforcement times its sectional area, regardless of the grade of reinforcement used.

The strength of the plain columns tested with slow loading was generally slightly less than that of the columns tested with fast loading. For the reinforced columns the strength with slow loading was nearly equal to, but generally slightly greater than that for fast loading.

The indication of the tests is that the strength of the columns having longitudinal and spiral reinforcement was made up of (1) 85 per cent of the cylinder strength times the net area of the concrete within the outer circumference of the spiral, (2) the yield-point stress of the longitudinal bars times their area, and (3) the strength added by the spiral, which amounted to about 70 per cent of the strength added by an equal amount of longitudinal reinforcement of intermediate grade steel.

Progress Report on Column Tests at the University of Illinois

BY F. E. RICHART* AND G. C. STAEHLE†

INTRODUCTION

The Investigation—The following progress report gives results of tests made in the investigation of reinforced concrete columns at the University of Illinois. As stated in a previous report, the investigation is sponsored by Committee 105 of the American Concrete Institute and the experimental work has been divided between Fritz Engineering Laboratory at Lehigh University and the Materials Testing Laboratory at the University of Illinois. The work at the latter laboratory is under the administrative direction of Dean M. S. Ketchum, Director of the Engineering Experiment Station and Prof. M. L. Enger, Head of the Department of Theoretical and Applied Mechanics.

The testing program outlined by the Committee and reported a year ago[1] has been followed quite closely, variations from it consisting principally of the addition of a few columns of plain concrete, and a small group, Series 8, to bring out the effect of long-continued loading on columns in which the reinforcing bars do not end flush with the end of the column. The time schedule followed in the testing program is given in Table 1.

This report contains results of the tests of Series 1 and 2, in addition to general information pertaining to all of the tests. The data of Series 3, in which the columns are being held under design load for one year before being tested to failure, are naturally incomplete; however, the strains measured over a five-months period of continued loading were felt to be of sufficient interest to warrant an early progress report, to appear in the March

*Research Associate Professor, Theoretical and Applied Mechanics, University of Illinois, Urbana.

†Associate Engineer, Structural and Technical Bureau, Portland Cement Association, Chicago.

[1]See Progress Report of Committee 105, Proc. A. C. I. Vol. 26, p. 601, 1930.

TABLE 1—TIME SCHEDULE OF TESTING PROGRAM

All dates given are for 1930 unless otherwise noted and are inclusive.

Operation	Series Number						
	1	2	3	5	6	7	8
Making columns	Jan. Feb.	April Aug.	April July	June Aug.	July Aug.	Aug. Dec.	Dec. Jan. '31
Applying continuous design load			June Sept.				Jan. '31 Feb. '31
Shrinkage and flow measurements			June Sept. '31				Jan. '31 Feb. '32
Loading to failure	Feb. Mar.	June Oct.	June '31 Sept. '31	Aug. Oct.	Sept. Oct.	Oct. Feb. '31	Jan. '32 Feb. '32
Number of columns	10	104	153	20	14	20	12

TABLE 2—TENSILE STRENGTH OF MORTAR BRIQUETS

Made with Lehigh Portland Cement, Standard Ottawa sand, 1:3 by weight.

Date of Making, 1930	Tensile Strength, lb. per sq. in.	
	Age, 7 Days	Age, 28 Days
March 4	316	408
April 25	261	318
June 3	314	376
June 25	294	372
September 5	316	398
Average	300	374

TABLE 3—SIEVE ANALYSES OF AGGREGATES

Values given are average of several analyses

Aggregate	Percentage of sample passing Sieve No.									Fineness Modulus
	100	50	30	16	8	4	⅜"	¾"	1½"	
Sand	3	15	51	73	88	98	100			2.72
Gravel			0	1	1	1	34	78	100	6.85

JOURNAL of the Institute. A similar report will be prepared soon for Series 5 and 6, while the tests of Series 7 and 8 are still unfinished.

1. *Type of Test Column*—All of the columns tested (except two of Series 1) were cylindrical, with a length seven and one-half times the core diameter. In all but those of Series 6, the fireproofing was omitted and the spiral reinforcement was practically at the column surface. In Series 6, a 2-in. thickness of fireproofing was used. As the result of the tests of Series 1, all columns except those of Series 8 were made with the milled ends of reinforcing bars flush with the concrete bearing faces. In Series 8, the ends of bars were stopped ¼ in. short of the end of the column, so that no bearing was transmitted to the bars.

2. *Materials*—The cement used in making columns was furnished by the Lehigh Portland Cement Co. It was stored in wooden barrels in a dry room until used. The strength, as shown by mortar and concrete tests, was uniformly high. Table 2 gives data of briquet tests made at different times during the season. From other tests made from time to time, the water required for normal consistency of the cement was found to be 23.0 to 24.0 per cent, by weight; the average time of initial set was 3 hr. 40 min.; of final set, 6 hr. 40 min. Tests of fineness showed 12 to 14 per cent retained on a No. 200 sieve after 10 min. in a Rotap sieve shaker. All soundness tests gave satisfactory results.

The torpedo sand used was furnished by the Lincoln Sand and Gravel Co., Lincoln, Illinois, and the gravel was furnished by the Neal Sand and Gravel Co., Mattoon, Illinois. Average sieve analysis and fineness moduli are given in Table 3. The aggregates used in Series 1, which differed from the above, are described in the section referring to that series.

The concrete mixtures used were designed to produce cylinder strengths of 2000, 5000 and 8000 lb. per sq. in. at 56 days, except in Series 1, wherein the tests were made at 28 days. A workability as shown by a slump of 3 to 6 in. was chosen, except in the case of the 8000-lb. concrete, which was designed to be placed with the aid of a high frequency electric vibrator, loaned for the work by the Electric Tamper and Equipment Co., Chicago, Ill. The proportions finally used are given in Table 4. The results of

TABLE 4—DATA OF CONCRETE MIXTURES

Materials: Lehigh portland cement; Lincoln, Ill., sand; Attica, Ind., gravel. Unit weight of aggregates, A. S. T. M. method, dry, rodded: sand 110 lb. per cu. ft., gravel 100 lb. per cu. ft.

Designed Strength lb. per sq. in.	Series Number	Proportions by Weight Cement, Sand, Gravel	Water-Cement Ratio	
			By Volume	Gallons per Sack
2000	2, 3, 7	1 : 4.05 : 4.95	1.40	10.5
	5, 6	1 : 4.05 : 4.95	1.40	10.5
		1 : 3.83 : 4.67	1.40	10.5
		1 : 4.18 : 5 27*	1.50*	11.25*
3500	2, 3, 7	1 . 3.27 : 3.98	1.10	8.25
	5, 6	1 : 3.27 : 3.98	1.10	8.25
		1 : 3.15 : 3.85	1.10	8.25
5000	2, 3, 7	1 : 2.25 : 2.75	0.80	6.0
8000	2	1 : 1.01 : 1.24	0.50	3.75

*Used in 28 and 32-inch columns to reduce ultimate column strength.

56-day tests of companion cylinders made with all columns are reported with the results of the various column tests.

The reinforcing bars were furnished by the Kalman Steel Co., Chicago, Ill., the Truscon Steel Co., Youngstown, Ohio, and the Rail Steel Bar Association, Chicago, Ill. The spiral stock was furnished by the American Steel and Wire Co., Chicago, Ill., and fabricated into spirals by the American System of Reinforcing, of Chicago. While complete tests of reinforcing steel have not been made, average properties of the various lots as determined to date are given in Table 5. It is planned to test coupons which will indicate the properties of the steel in each individual column.

The reinforcement used in Series 1, made before the main stock of steel was received, differs slightly from the remainder in the above list. The spirals of cold drawn wire were intended to approximate the quality to be obtained in a hot rolled rod of intermediate grade. The longitudinal steel was nearly the same as that received later and used in the remaining series.

3. *Details of Reinforcement*—The 8-in. columns were designed to have three percentages of longitudinal steel, 1.5, 4, and 6, and two percentages of spiral, 1 and 2. The larger columns were

TABLE 5—AVERAGE TENSILE PROPERTIES OF REINFORCING STEEL

Nominal sizes of bar used in all computations.

Bar Size in.	No. of Tests	Grade of Steel	Yield Point* lb. per sq. in.			Ultimate Strength lb. per sq. in.	Per Cent Elong. in 8 In.	Used in Series No.
			Max.	Min.	Av.			
				Longitudinal Steel				
½ rd.	24	Int.	50,600	43,800	45,600	73,300	19.5	2, 3
½ sq.	3	Int.	48,300	47,700	47,900	74,900	22.3	1
½ sq.	182	Int.	57,000	50,800	53,400	83,200	21.4	2, 3, 6, 7
½ sq.	26	Struct.	42,100	36,700	40,400	59,300	28.2	2
½ sq.	24	Rail	75,400	63,000	68,300	109,100	16.0	2
⅝ rd.	22	Int.	43,600	35,800	39,300	61,800	27.1	2, 3, 7
¾ rd.	32	Int.	54,000	48,600	51,100	84,500	21.2	2, 3, 7
⅞ rd.	45	Int.	43,300	40,200	41,700	69,700	26.0	5, 6
1 rd.	16	Int.	56,200	48,700	50,400	83,900	23.3	5, 6
1⅛ sq.	8	Int.	46,300	43,400	45,300	79,000	19.0	5, 6
				Hot Rolled Spiral Rod				
No. 5	19	Int.	53,000	44,500	49,400	79,500	14.5	2, 3, 6
¼	69	Int.	52,800	39,700	48,200	74,200	19.0	3, 5, 6
5/16	10	Int.	47,100	39,700	44,500	75,500	20.0	5, 6
⅜	25	Int.	45,200	38,000	41,300	71,000	22.6	5, 6
				Bright Cold Drawn Wire				
3/16	2	Wire	49,000	47,000	48,000	81,900	....	1
No. 5	6	Wire	85,200	75,300	80,400	90,000	3.0	7
¼	6	Wire	90,000	74,500	83,200	109,400	4.3	7

Rail steel bars were plain sq., all others deformed.

*No yield point on spiral stock. Values given represent a "useful limit" of the material, and are stresses at a unit elongation of 0.005. This elongation is based upon observations of spiral strains at the ultimate load on columns made with various percentages of spiral reinforcement. (See Bulletin 190, Univ. of Ill. Eng. Expt. Sta.)

designed for 1.5 or 4 per cent of longitudinal steel and 1 per cent of spiral. The commercial sizes chosen to meet these designed values are given in Table 6, to which reference may be made in connection with the tabulated results of all tests. The number of bars selected are in multiples of 4, to facilitate strain measurements on four sides of each column. It is seen that the largest amount of longitudinal steel used in the 28-in. columns was 3.28 per cent. The original design of 4 per cent was reduced in this case to obviate the chance of exceeding the capacity of the 3,000,000-lb. testing machine. Another variation from the design percentage is seen in the 1.24 per cent spirals for 8-in. columns. With the smallest size of hot-rolled spiral rod available this was the smallest percentage possible without exceeding a pitch of one-sixth of the core diameter.

TABLE 6—DETAILS OF REINFORCEMENT USED IN TEST COLUMNS

Nominal sizes of reinforcement are used in all calculations.

Core* Diam. in.	Height ft. - in.	Longitudinal Bars			Spirals			
		No. and Size	Per Cent	Used in Series No.	Size	Pitch in.	Per Cent	Used in Series No.
8	5 - 0	4 - ½″ rd. 8 - ½″ sq.	1.57 3.98	2, 3 1,2,3,6,7	3/16″ No. 5 (.207″)	1.35 1.35	1.00 1.24	1 2, 3, 6, 7
		4 - 5/8″ rd. 4 - ¾″ rd.	5.94	2, 3, 7	¼″	1.19	2.00	3, 7
12	7 - 6	4 - ¾″ rd. 8 - ⅞″ rd.	1.56 4.25	5, 6 5, 6	¼″	1.60	1.00	5, 6
20	12 - 6	8 - ⅞″ rd. 16 - 1″ rd.	1.53 4.00	5, 6 5, 6	5/16″	1.51	1.00	5, 6
28	17 - 6	12 - 1″ rd. 16 - 1⅛″ sq.	1.53 3.28	5, 6 5, 6	⅜″	1.56	1.00	5, 6

*Percentages of reinforcement based on this core diameter, out to out, of spiral.

4. *Making and Storage of Columns*—The concrete for all columns was mixed in a one-bag batch mixer, the usual mixing time being 4 minutes. For the 8-in. columns all aggregates were reduced to a room-dry condition previous to use. Cement, aggregates and water were proportioned and measured out by weight. The 8-in. columns were made from a single batch, with sufficient excess to permit the making of six companion 6 by 12-in. cylinders. The concrete was wheeled to the forms in wheelbarrows and puddled with the aid of a steel tamping rod and a light air-hammer used on the outside of the form. In pouring the larger columns, each batch was hauled to the form in a bottom-dump bucket mounted on a truck, lifted by a 10-ton crane, and discharged into the form. The largest columns required about 25 batches of concrete and weighed about 8 tons.

The 8 and 12-in. forms were made of steel pipe, split into quadrants and held together with circular clamps. The larger forms were standard steel column forms, loaned by the Kalman Steel Co. for the purpose.

The vertical bars used in all columns were milled to exact length, then wired to the spirals to form a unit or cage. The unit was placed on a machined base plate, the form assembled around it and plumbed. To insure that the ends of bars remained in a plane, a machined 6-in. pipe flange (for the 8-in. columns) was forced and held down against the upper ends of the bars, and the concrete placed in the column through the 6-in. opening in the flange. For the larger columns it was found satisfactory to assemble the reinforcing unit in a vertical position on the base plate before wiring, with the result that the weight of the bars held them in contact at the bottom. In placing the concrete in the columns, the concrete was brought up to about even with the tops of the reinforcing bars. After the concrete had stiffened, a thin layer of neat cement paste or 1:1 mortar was applied to the top and a machined bearing plate pressed down on this cap until it rested firmly on the milled ends of the bars. With this technique it was not difficult to bring the ends of bars into the plane of the cement cap.

Forms were removed from the 8-in. columns the day after moulding, gage lines were prepared by exposing the reinforcing steel and by setting small steel plugs in the concrete with quick-setting cement. Gage holes were drilled for strain measurements with a No. 54 drill. Initial strain readings were taken and the columns were placed in the moist room for the remainder of the 56-day moist storage period. The air of the moist room was kept saturated by humidifying sprays and the temperature was held at 70 deg. F. with an average weekly variation of about one degree. Due to the size and weight of the large columns, it was impossible to place them in the moist room. They were stored in the main crane bay of the laboratory, under burlap which was wet down generally three times a day. The temperature of the room during this curing period—June to September—ranged from about 70 to 90 deg. F. The temperature beneath the wet burlap was considerably lower and the average value did not vary greatly from 70 deg. F.

5. *Testing Methods*—All columns except those of Series 3 were tested to failure in one of four testing machines, a 3,000,000-lb. Southwark-Emery hydraulic machine, a 600,000-lb. Riehle, a

TABLE 7—PRECISION OF STRAIN GAGES

Gage	Gage Length in.	Dial Graduation in.	Mult. Ratio	Estimated Precision* Millionths of inch per inch
Whittemore	10	0.0001	1	6
Berry No. 2040	10	0.001	5	10
Berry No. 2040	8	0.001	5	12
Berry No. 2280	8	0.001	5	8
Diameter gage	8	0.001	5	15
Howard type	10	0.001	1	150
Howard type	50	0.001	1	20

*Based upon general working condition of instrument.

300,000-lb. Riehle, and a 300,000-lb. Olsen screw-power testing machine. A rate of loading was chosen for the hydraulic machine which was comparable with the speeds obtained with the screw-power machines.

Load was applied to the columns through a top spherical bearing block, and in some cases through both top and bottom blocks. With the exception of tests of Series 2, after a small zero load was applied, these blocks were wedged in a fixed position, but it is quite evident that after any considerable load is applied such a block is held against rotation by friction alone. The columns were thus loaded as "flat-ended" columns.

Strain measurements were made by the use of various types of gages. In the majority of the work, longitudinal strains were measured with a 10-in. Whittemore gage at gage lines on four sides of the column on concrete and on steel. The centers of these lines were located 10 in. from the top and bottom of the column, and at midheight. Lateral deformations were measured by use of a "diameter" gage which indicated changes on diametral gage lines. Other gages were a 10-in. Berry gage, 8-in. Berry gages with long legs which were required on the columns having 2 in. of fireproofing, a 50-in. direct reading gage which was used to measure volume changes during moist storage, and a 10-in. direct reading gage for measuring large strains beyond the range of the Whittemore and Berry gages. A summary of the constants of the gages used and their probable precision is given in Table 7.

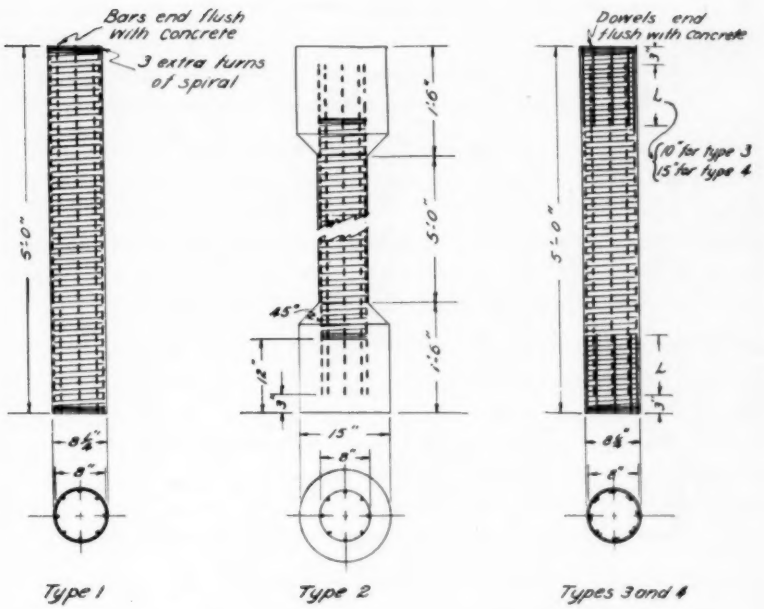


FIG. 1—TYPES OF COLUMN USED IN SERIES 1

SERIES 1—STUDY OF END CONDITIONS

6. *Results of Tests of Series 1*—This group of ten columns was designed to furnish information as to the most desirable type of end condition to be used in the columns of the later series of tests. Four types of end condition are included in the group, two columns of each type, except Type 1 for which there were four. Fig. 1 gives details of the four column types. All of the columns had eight ½-in. square bars (4 per cent) of intermediate grade, and $\frac{3}{16}$-in. cold drawn spirals, 8 in. outside diameter, and 1.35-in. pitch (1.0 per cent). The columns of Type 1 had the ends of reinforcing bars flush with the ends of the concrete. In Type 2, a capital 15 in. in diameter and 18 in. long was added at each end of the column shaft, making the total length 8 ft. The reinforcing bars extended into these capitals to a point 3 in. from the bearing faces, but the spirals were stopped 12 in. from the ends of the

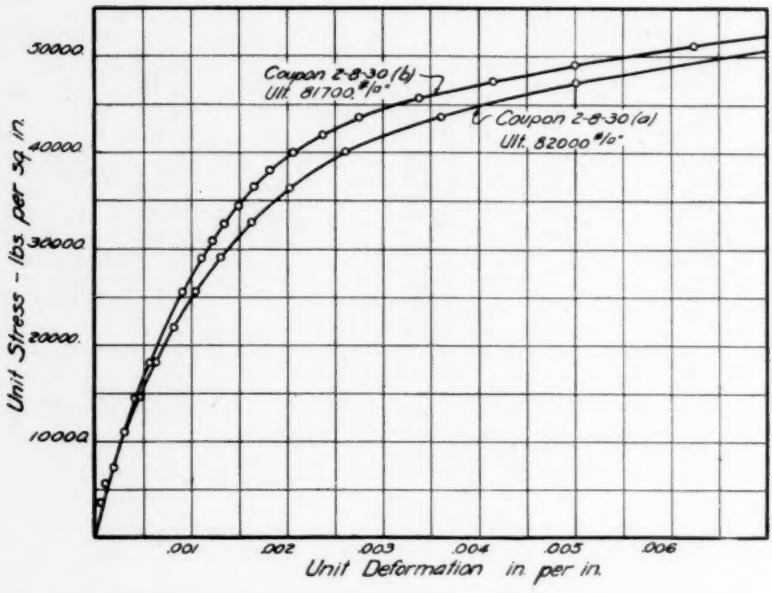


FIG. 2—STRESS-STRAIN CURVES FOR SPIRAL REINFORCEMENT, SERIES 1

column. No other reinforcement was used in the capitals. In Type 3 a 20-diameter splice was made in the longitudinal bars at each end of the column. This was made by stopping the main bars 3 in. from each column end and using 13-in. dowel bars which ended flush with the column ends. Type 4 was like Type 3 except that a 15-in. lap (30 diameters) was used. The dowels were wired to the main longitudinal bars.

Properties of the reinforcing steel are given in Table 5. The cold-drawn wire used in this series did not have a definite yield point; typical stress-strain curves are given in Fig. 2.

The concrete for these columns was intended to have a compressive strength of 3500 lb. per sq. in. at 28 days, but due mainly to the fact that no trial mixtures had been made with the cement used, the concrete obtained gave an average cylinder strength of 5195 lb. per sq. in. The concrete was made with different aggregates from those used in later tests, a torpedo sand from

Attica, Indiana, having a fineness modulus of 3.03 and a ¾-in. Attica gravel having a fineness modulus of 7.21. With the water-cement ratio used, 0.90, slumps of 2.8 to 6.3 in. were obtained. The proportions used, by weight, were 1:2.5:2.8.

The moulds were made from 8-in. steel pipe, cut into quadrants. To accommodate the three spacing bars on the spirals the mould diameter was increased slightly by the introduction of ¼-in. fillers between segments. Leakage of water from the concrete of these specimens led to the practice in later series of tests of sealing all joints with paraffin previous to casting of columns.

Following a procedure outlined by Committee 105, after the forms were stripped, the columns were given two coats of linseed oil before they were placed in moist storage. This was intended to prevent loss of moisture during temporary exposure to the air while gage lines were being prepared and also at the time of testing. However, in the later series this procedure was omitted as unessential and of doubtful value.

The columns were tested when 28 days old, in the 600,000-lb. Riehle testing machine. A spherical bearing block was used at top and bottom of all columns except the two having enlarged ends, with which only the upper bearing block was used. After columns were plumbed and a small initial or "zero" load was applied, the spherical blocks were wedged against further movement. After the initial load of about 6000 lb., a full set of strain readings was taken. The remaining load was applied in about 10 equal increments, and a complete set of strain readings was taken at each increment.

As previously noted, the strain readings were taken at three levels, at middle and 10 in. from each end of the column. In the columns with lap splices in the reinforcement, readings were taken only on the main bars; none on the short dowel bars. Fig. 3 shows views of columns of Types 1 and 2 during testing.

Since the spiral wire was of relatively low strength, approximating intermediate grade hot rolled rod, and since the concrete was of relatively high strength, the one per cent of spiral did not add a very large margin of strength beyond the yield point of any of the columns of Series 1. Little lateral deformation was

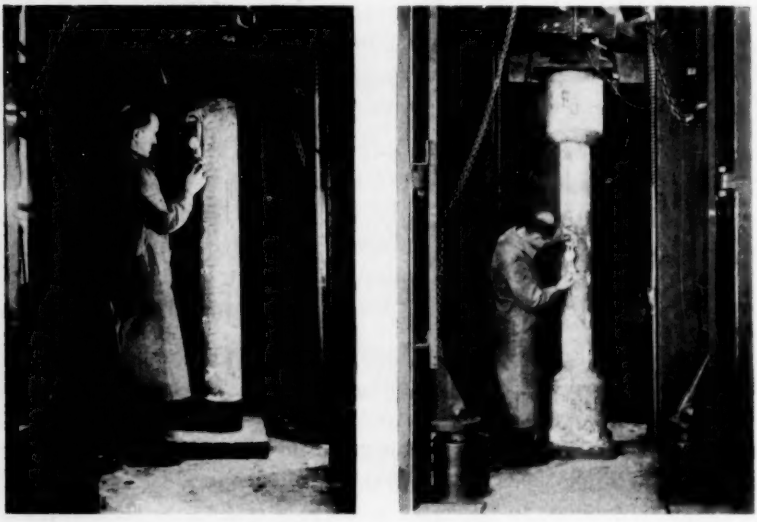


FIG. 3—COLUMNS OF TYPES 1 AND 2 DURING TEST, SERIES 1

noted until the yield point of the column was reached and no appreciable spalling occurred until shortly before the ultimate load was applied. The spiral wire did not break until the maximum load had been passed; then failure occurred by the snapping of one or more wires and outward buckling of the longitudinal bars.

The manner of failure of the two columns of Type 2, with capitals, is noteworthy. Column *g* failed in the shaft in much the same manner as the other columns but breakage of spiral wire after failure was forestalled by splitting of the capital at the top of the column. The strength of this column was equal to that of the strongest column of Type 1. Column *j* failed through the splitting of both capitals, apparently before the full strength of the shaft had been developed.

The principal results of the tests of Series 1 are given in Table 8, together with data on the quality of the concrete used. Both columns and companion cylinders showed unusual uniformity of strength. It is surprising that there was only 5 per cent difference

TABLE 8—RESULTS OF TESTS OF SERIES 1

Reinforcement 8½ in. squares, intermediate grade; Spirals, $\frac{3}{16}$-in. cold drawn wire, 1.35 in. pitch. Spirals 8 in. outside diameter, Core area 50.2 sq. in. Cylinder strengths represent average of three tests. Age of columns and cylinders at test, 28 days.

Type	Col. No.	End Conditions	Maximum Load			6 by 12-in. Cyl.	
			lb.	lb. per sq. in.	Per Cent of Type 1	Slump in.	Comp. Strength lb. per sq. in.
1	a	Bars milled, ends flush with concrete	376,300	7500		6.3	5080
	b		367,000	7300		6.3	5080
	h		358,700	7150		4.2	5220
	k		374,100	7450		4.4	5310
	Av.		369,200	7350	100.0	5.3	5170
2	g	Enlarged capitals at ends	376,900	7510		4.4	5220
	j		347,000	6910		4.2	5310
	Av.		361,900	7210	98.1	4.3	5265
3	c	20-diam. lap of bars. Dowels milled and flush at ends	331,000	6600		2.8	5240
	d		375,300	7480		2.8	5240
	Av.		353,200	7040	95.6	2.8	5240
4	e	30-diam. lap of bars. Dowels milled and flush at ends	365,300	7280		6.3	5110
	f		345,300	6880		6.3	5110
	Av.		355,300	7080	96.2	6.3	5110

in average strength between the columns of Type 1, which were the highest, and those of Type 3, which were the lowest of the group. It will also be noted that the average compressive strength for all of the companion cylinders was 5195 lb. per sq. in. and that the variations in the concrete strengths for the four different types of columns did not exceed 1.6 per cent from this average value. Considering that the concrete furnished roughly 60 per cent of the strength of the column, it will be seen that the variations in concrete quality should not produce a difference in the strength of the different types of column amounting to more than 1 per cent. Accordingly no attempt has been made to correct the relative strengths of columns to allow for these minor variations in quality of concrete.

Various attempts have been made to account for the portion of the strength of a column contributed by each of its elements, and most column formulas indicate the share of the load carried on concrete, on vertical reinforcement and on spiral reinforcement. It is evident that the results of Series 1, comprising only 10 column tests and in which variable end conditions were introduced, should not be used to establish any general column formula. The object of the following tabulation is merely to present the results in a form that may easily be studied when the results of the later and more comprehensive series of tests are available. A well-established method of expressing the strength of a column is to consider it made up of three elements, (1) the ultimate strength of the net concrete core area, considered as plain concrete, (2) the load required to stress the compressive reinforcement to the yield point, and (3) the load required to stress the spiral to its limit of effectiveness. It is definitely known that the strength of the concrete in a column seven and one-half diameters in length should be less than that of a 6 by 12-in. cylinder. Tests in this laboratory in 1924 and 1925 showed the strength of 8 by 40-in. and 10 by 40-in. plain columns to be about 85 per cent of that of companion cylinders; the results of Series 2 for 8 by 60-in. plain column (see Table 10, to follow) show a column-cylinder strength ratio of about 0.80. The corresponding tests of Series 2 made at Lehigh University show a ratio of about 0.85 and in the present studies this value will be used. Since no plain columns were included in Series 1, an estimate of the load carried by the concrete portion of the column will be taken at 85 per cent of the companion cylinder strength. The longitudinal steel consisted of 3.98 per cent of intermediate grade, having a yield point of 47,900 lb. per sq. in. There is no definite yield point of the spiral steel; however, an estimate of the useful limit of strength of spiral may be made on the basis of previous tests,[1] in which the spiral strain at the ultimate column load was found to vary from 0.003 to 0.009. Accordingly, the stress in the spiral wire at a strain of 0.005 has been chosen as a rough measure of the useful limit of the spiral reinforcement; the average value found being 48,000 lb. per sq. in. Table 9 gives the distribution

[1]Bulletin 190, Engineering Experiment Station, Univ. of Illinois.

TABLE 9—ANALYSIS OF DISTRIBUTION OF COLUMN STRENGTH, SERIES 1

Type of Col.	End Condition	Column Strength P/A lb. per sq. in.	Cylinder Strength f'_c lb. per sq. in.	Portion Col. Str. Attributed to		
				Concrete*	Vert. Steel	Spiral Steel
				$0.85f'_c$ x 0.96 lb. per sq. in.	(3.98 per cent f_s = 47900) lb. per sq. in.	(1.0 per cent f_s = 48000) lb. per sq. in.
1	Ends flush	7350	5170	4215	1900	1235
2	Capitals	7210	5265	4290	1900	1020
3	20-d lap	7040	5240	4270	1900	870
4	30-d	7080	5110	4165	1900	1015
Average		7170	5195	4235	1900	1035

*Concrete section is 96 per cent of core area.

of the strength of the columns of Series 1 on the basis of the above method of calculation. It must be remembered that this calculation has thrown most of the variability of the column strengths into that portion of the strength attributed to the spiral reinforcement, whereas it is much more probable that such variations were due to the different arrangements of the vertical steel.

The longitudinal strains measured on all columns of the series are shown on the load-strain curves of Fig. 4. The strains in concrete and steel at the different elevations are seen to be similar, except for the columns of Types 3 and 4, in which dowels were used. In these columns, the strains measured near the ends of the main reinforcing bars are generally less than those in the adjoining concrete. This is because the gage line is within the lap splice in the reinforcement.

The variation between strains at top, middle and bottom of each column may be seen in Fig. 4. While the point at which high early strains developed usually coincided with the point of failure, no consistent rule may be stated for the location of the point of failure in the columns. In those with dowels, failure occurred within the zone of the reinforcement splice, and in the others the greater number of failures were in the upper third of the column shaft.

The modulus of elasticity of the concrete used in the columns, as found from tests of 6 by 12-in. cylinders, averaged 4,320,000 lb. per sq. in. for the 10 columns. The lower portion of the stress-

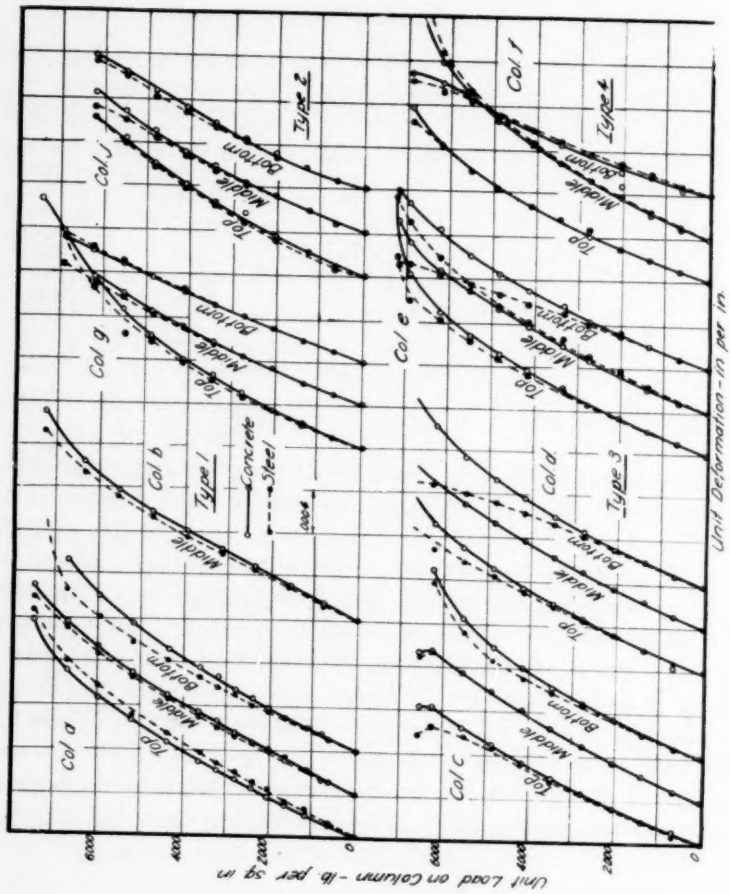


FIG. 4—STRESS-STRAIN CURVES FOR COLUMNS OF SERIES 1

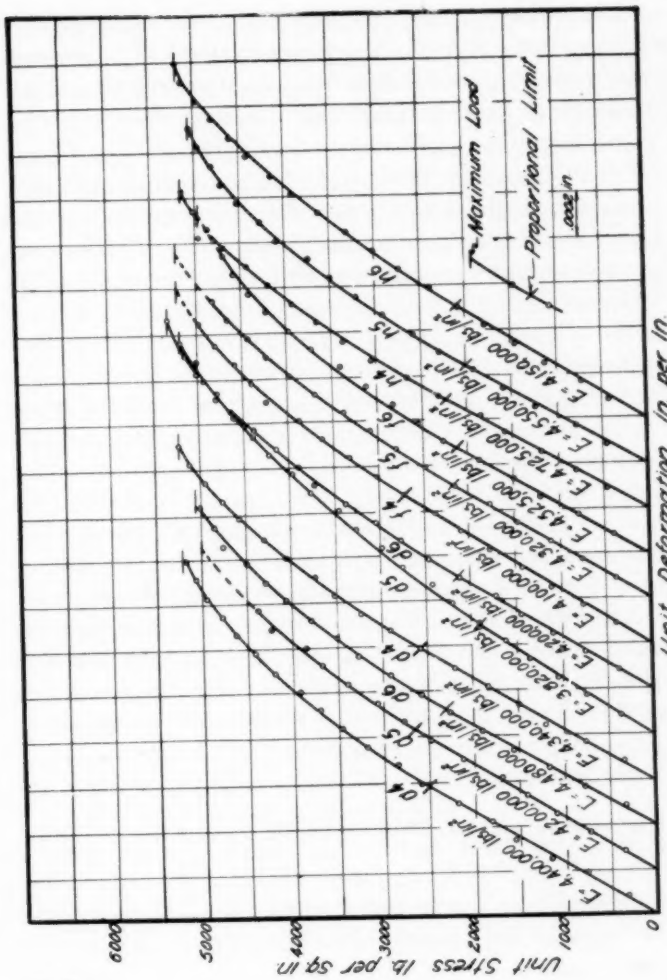


FIG. 5—STRESS-STRAIN CURVES FOR CONCRETE OF SERIES 1

strain curve for these cylinders was a straight line. The limit of proportionality for the concrete was found to be 37 to 54 per cent of the ultimate strength, with an average of 44 per cent. Fig. 5 shows typical stress-strain diagrams for this concrete.

From a consideration of the relative strengths of the columns of the four types given in Table 8, it might be concluded that any one of the four types would give satisfactory results with the possible exception of the type having enlarged ends. However, since there was no apparent advantage with either the type having capitals or those having dowels, economy of material and labor pointed to the selection of Type 1. This type was used in all of the remaining columns of the investigation with the exception of a few columns of Series 8 which were designed to show the effect of long-continued loading on the transfer of stress to the longitudinal reinforcement by bond alone. It might be noted that the amount of material required for the type with capitals was roughly twice as great as for the one finally chosen, and that the labor of making and testing such columns and the moist storage room required for them would be much more than twice as great. The use of dowel bars showed no particular advantage or increase in information to be secured, and an objection to their use was found in the difficulty of securing proper contact of the milled ends against the bearing plates. The wisdom of the choice of Type 1 for the succeeding series of tests has been borne out in the conduct of the later series.

SERIES 2—STUDY OF METHOD OF LOADING

7. *Results of Tests of Series 2*—This series of tests was planned to give information on a number of questions relating to column strength, including different methods of loading, a considerable range in kind and amount of reinforcement and in quality of concrete. The columns were made in general with three grades of concrete, designed to produce 56-day strengths of 2000, 3500 and 5000 lb. per sq. in. In addition a small group was made with concrete designed to produce a 56-day strength of 8000 lb. per sq. in., when compacted into place by the use of an electric vibrator.

Longitudinal reinforcement of three different grades was included in the series and in general three percentages ranging from

1½ to 6 per cent were used. The grades of steel used were specified to meet requirements for intermediate, structural, and rail steel. The spiral reinforcing used was of intermediate grade, hot rolled rod; about 1¼ per cent was used in all columns of this series.

As noted, one of the principal studies embodied in the series was that of rate of loading of the test column. The method used in practically all previous column tests has been to apply the load in increments, allowing only sufficient time between increments for the observation of strains. It was felt that in the range of plastic action of the column a longer period of time between increments would produce a considerable amount of plastic deformation and might quite possibly have a decided effect upon the carrying capacity of the column. Accordingly, two methods of testing were adopted. These will be denoted in the following discussion as "fast" and "slow" loading. Under the "fast" loading the load is applied to the column in 10 or more increments with a time interval between increments of 15 minutes or less, during which strain measurements were taken. In the "slow" loading, a load of approximately ¾ of the estimated yield point strength of the column was applied in 8 to 10 equal increments at the same rate of loading as used in the fast loading method. Subsequent loads were applied in 6 to 10 increments at intervals of four hours. With this procedure, strain readings were taken during each 4-hr. interval immediately before and after the application of an increment of load.

The amount of flow in the test piece during a 4-hr. period naturally caused a certain amount of release of load in the testing machine. To minimize this condition for the tests under slow loading, a special load-holding device was used. This consisted of a bed of coiled springs having a total capacity equal to or greater than the column strength and having a total shortening of approximately 1½ in. The shortening of a column due to plastic yielding of a few thousandths of an inch produced very little difference in the pressure exerted by this bed of springs and hence the load was held nearly constant throughout a full 4-hr. period. A view of a column as tested with one of these devices is shown in Fig. 6. Another feature of the tests with slow loading, which lasted from 36 to 48 hours, was the use of a protective case

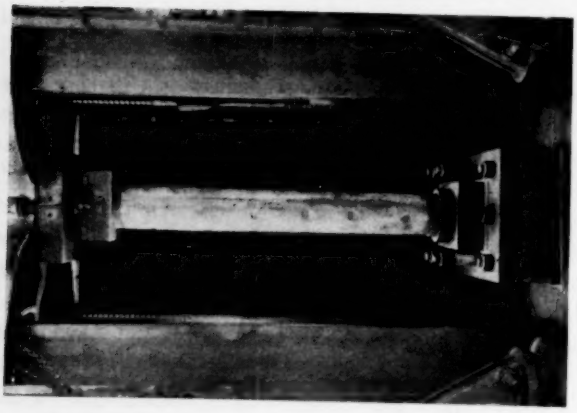


FIG. 7—PROTECTIVE CASE USED WITH SLOW LOADING TESTS, SERIES 2

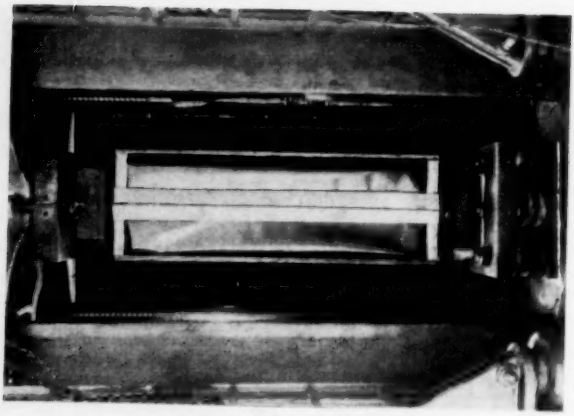


FIG. 6—COLUMN ON TESTING BASE, SLOW LOADING, SERIES 2

which was placed around the column to prevent its drying out. Such a case, lined with wet burlap, is seen in the view of Fig. 7.

The tabulated results of all of the tests of Series 2 are given in Table 10. In all but the tests in which 8000-lb. concrete was used, there were companion specimens tested under both the fast and slow loading, two columns of a kind. In addition to the columns scheduled in the 1930 committee report previously mentioned, 27 plain columns were also made and tested to give further information on the action of the concrete alone under fast and slow loading.

The data of Table 10 indicate that there was not a very marked difference in the carrying capacity of columns tested under the two methods of loading. There was, however, a considerable difference in the amount of deformation measured as indicated by the typical stress-strain curves for corresponding gage lines of columns tested under fast and slow loading, as shown in Fig. 8. From these results it appears that the amount of flow which takes place during a 4-hr. period, while producing quite appreciable changes in length of the test piece, has not produced appreciable changes in strength, except perhaps for the plain columns. The difference in action between plain and spirally reinforced columns is quite readily explained. As has been found in previous investigations, after plain concrete has reached 75 to 85 per cent of its ultimate strength, a decided change in its action takes place wherein lateral bulging develops, internal breakdown of the structure begins, and plastic deformation increases rapidly. The failure of plain concrete may then be expected to take place sooner under the slow loading than under the fast loading. When concrete is restrained within an envelope of spiral reinforcement, it soon reaches a stage of almost entirely plastic deformation wherein the loads carried are in a definite proportion to the lateral pressure exerted by the spiral reinforcement. This proportion is independent of the amount of lateral deformation and hence the column may be expected to carry a given load as long as a definite pressure is exerted by the spiral reinforcement. Failure evidently occurs when the spiral becomes unable to furnish increased lateral pressure with an increase in lateral deformation. Whether it would ever be possible to have sufficient plastic de-

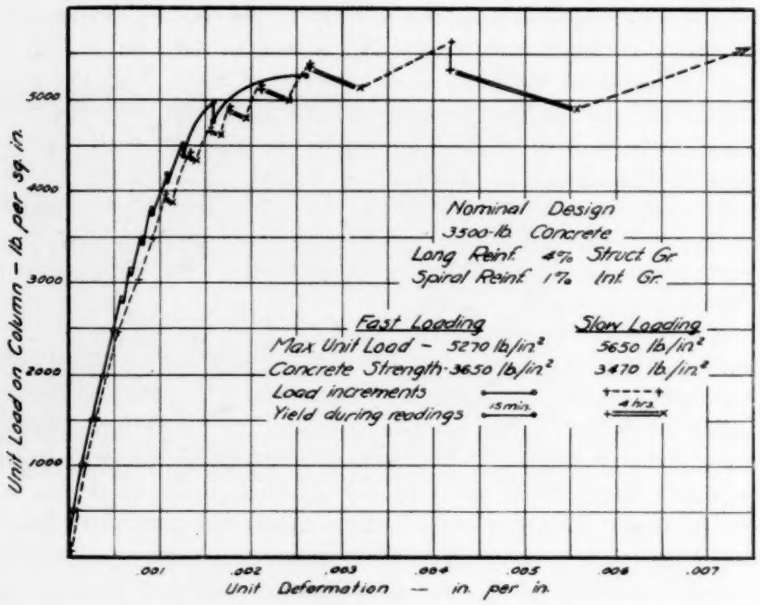


FIG. 8—STRESS-STRAIN CURVES FOR FAST AND SLOW LOADING, SERIES 2

formation or flow of the concrete between spiral wires to cause a decrease in the pressure exerted by the spiral reinforcement is problematic. The relation between the deformations of columns under fast and slow loading will be discussed further in a later report.

A comparison of ultimate column strengths obtained under fast and slow loading methods may be made by reference to Fig. 9, in which column strengths are plotted against the strength of concrete control cylinders. The figure contains the results from all columns of the series made with 1.24 per cent of spiral reinforcement and 3.98 per cent of longitudinal reinforcement of the three grades shown as well as the strength of the plain concrete columns. It is seen that there was no consistent difference in the strengths obtained by the two methods of testing; such differences as are found might be considered as within the range of variation

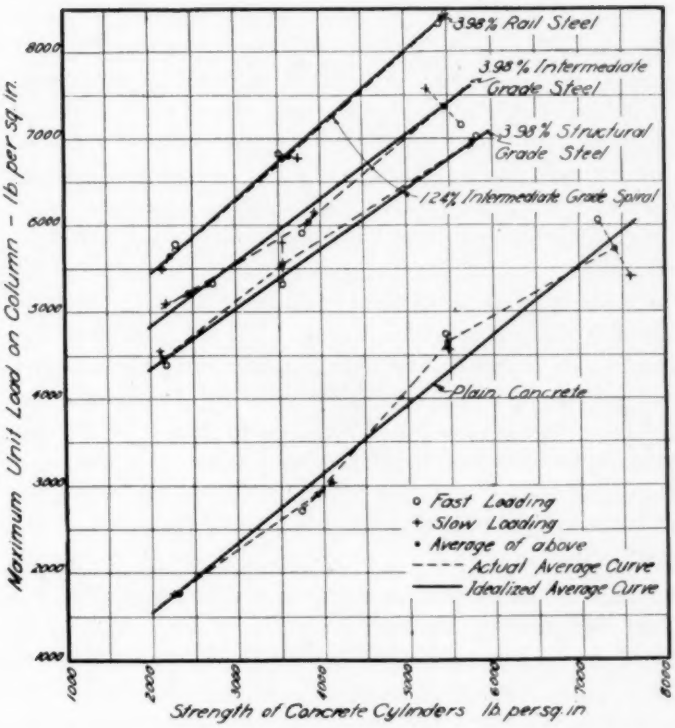


FIG. 9—RELATION BETWEEN COLUMN STRENGTH AND STRENGTH OF CONTROL CYLINDERS, FAST AND SLOW LOADING, SERIES 2

to be met had a single method of loading been used. For this reason, curves have been drawn to represent the average results from the two methods of loading. In plotting these curves the strengths of two individual columns, noted in Table 10 as having shown excessive deflection before failure, have been neglected. One of these columns was thrown clear out of the testing machine at failure, and both appear much weaker than their companion columns.

The curves of Fig. 9 also permit a study of the effect of the three grades of longitudinal reinforcement, and of the three concrete mixtures employed. The points for columns made with rail

steel fall upon a straight line which is practically parallel to the average curve for plain concrete columns in the lower part of the diagram. For columns made with bars of intermediate and structural steel, the plotted points are represented in two ways; the average points are connected by light dotted lines, which represent actual test results; and in addition, heavy solid lines are drawn which fit the respective sets of points reasonably well. In drawing these latter curves, less weight has been given to the points representing 3500-lb. concrete than to the others. It will be noted that the slopes of these two curves are slightly less than the one for plain concrete columns. The average slope for the four curves shown is slightly less than 0.8. This verifies previous tests which indicate that the difference in strength of spirally reinforced columns due to differences in concrete mixtures is equal to the difference in strength of plain columns of these mixtures, times the quantity $(1-p)$. That is, the slopes of the upper three curves might be expected to be 96 per cent of that for plain columns.

The intersections of the solid average curves of Fig. 9 with verticals representing concrete strengths of 2000, 3500 and 5000 lb. per sq. in. have been plotted in Fig. 10, correction being made in this way for variations in actual cylinder strengths from the designed values. Fig. 10 shows column strengths plotted against the average yield point stress for each grade of longitudinal reinforcement. Such a comparison is admittedly inaccurate since there was a considerable variation in the yield point stress for each grade of steel, but corrections for variations in the reinforcement in individual columns cannot be made until the tests of steel coupons are completed. (Test coupons have been saved from the two 22-ft. lengths of bar used in each column of this group.) Fig. 10 shows, however, that the increase in column strength is nearly proportional to the increase in average yield point stress for the reinforcement. With 3.98 per cent of reinforcement, an increase in yield point stress from 40,400 to 68,300 lb. per sq. in. should produce an increase in column strength of 0.0398 times 27,900 or 1110 lb. per sq. in., while the average increase shown for the three curves is 1300 lb. per sq. in. The rate of increase is nearly the same for the columns made with the three different grades of concrete.

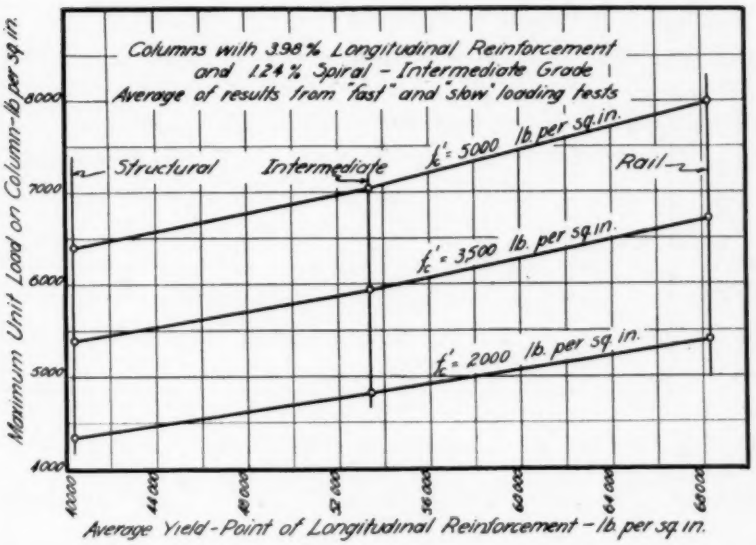


FIG. 10—RELATION BETWEEN COLUMN STRENGTH AND YIELD POINT STRESS FOR LONGITUDINAL REINFORCEMENT, SERIES 2

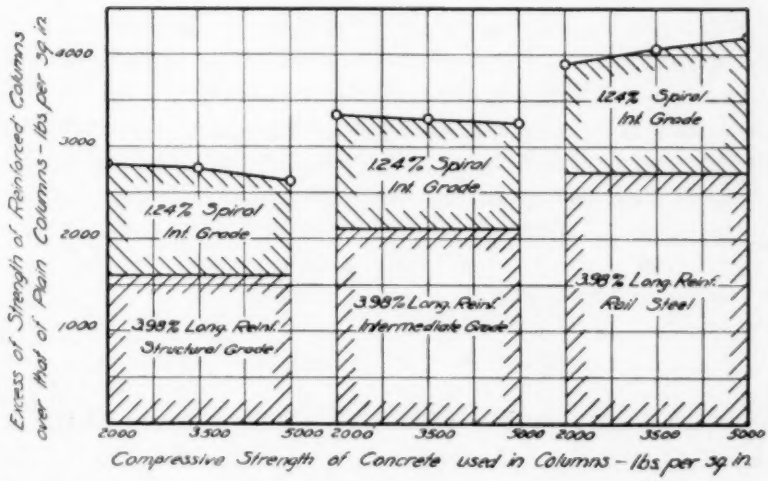


FIG. 11—CALCULATED DISTRIBUTION OF COLUMN STRENGTH BETWEEN LONGITUDINAL AND SPIRAL REINFORCEMENT, SERIES 2

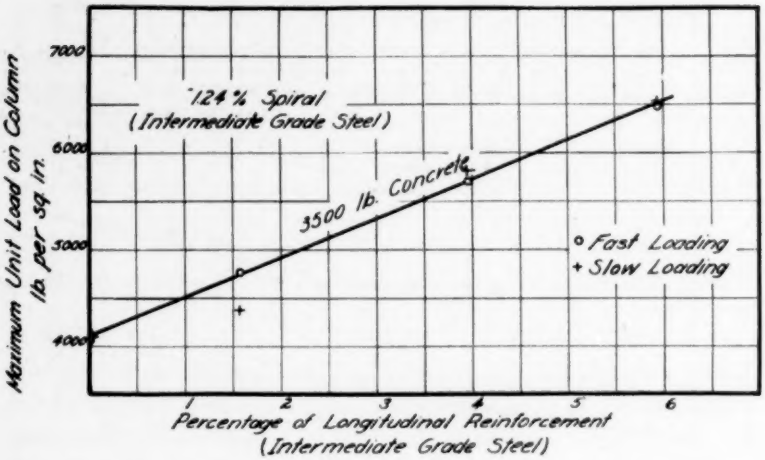


FIG. 12—RELATION BETWEEN COLUMN STRENGTH AND PERCENTAGE OF LONGITUDINAL REINFORCEMENT, SERIES 2

In studying the distribution of column load between the concrete section, the longitudinal steel and the spiral steel, further reference may be made to Fig. 9. Considering that the concrete of the reinforced columns carried a stress equal to the strength of the plain columns on a section 96 per cent of the core area, the portion of the column strength to be attributed to the reinforcement is found, and is shown by the plotted points in Fig. 11. This is distributed between longitudinal and spiral reinforcement, by computing the load on the former, 0.0398 times the yield point stress. The shaded areas indicate the relative portions of the column load attributed to each form of reinforcement. In view of the fact that accidental variations in column strength have thus been thrown into the portion attributed to the spiral reinforcement it is of interest that this portion is so nearly constant for the nine combinations of concrete and longitudinal steel shown.

Another group of columns of this series was made with 3500-lb. concrete, 1.24 per cent of intermediate grade spiral, and 0, 1.57, 3.98 and 5.94 per cent of intermediate grade longitudinal steel. The column strengths found with both fast and slow loading are

plotted in Fig. 12 against percentages of longitudinal reinforcement. Variations in concrete strength as shown by the control cylinders for these columns have been corrected by the use of the average curve for plain columns given in Fig. 9, and the column strengths shown in Fig. 12 correspond to cylinder strengths of 3500 lb. per sq. in. As in Fig. 9, there is little difference in the results for fast and slow loading. The test values are represented by an average curve for the two loadings that is nearly a straight line. Since there is some variation in the quality of the steel used to produce the three percentages, the average curve should not be exactly linear. The increase in strength with increase in longitudinal reinforcement is slightly less than that computed from the percentages and the average yield point stresses as listed in Table 5. The average yield point for the 1.57 per cent steel was 45,600 lb. per sq. in.; that for 3.98 per cent, 53,400 lb. per sq. in.; and for 5.94 per cent, 45,200 lb. per sq. in. On this basis the contribution of the three percentages of longitudinal reinforcement to the column strength should be 715, 2150 and 2685 lb. per sq. in. respectively. From Fig. 9, 3500-lb. concrete should contribute to the column strength about 2750 (1-p) lb. per sq. in. This leaves a remainder which may be attributed to the 1.24 per cent of spiral reinforcement averaging 1165 lb. per sq. in. for the four sets of test values.

Series 2 also included tests of columns made of concrete designed to give a 56-day cylinder strength of 8000 lb. per sq. in. The tests were made by the fast loading method only. The strengths obtained with this rich concrete were quite variable, and since a shortage of reinforcing spirals delayed the testing of part of the group until January, 1931, no analysis of the test results will be given here. The principal results for the group are given in Table 10.

Further analyses of the results of Series 1 and 2, on the basis of more complete information regarding the reinforcement, and in the light of the results of the remaining parts of the investigation, will be made in a later report.

TABLE 10—RESULTS OF TESTS OF SERIES 2

All reinforced columns 8 in. core diameter, 5 ft. long, 2 companion columns of each type Columns cured 56 days in moist room and tested moist. All spiral reinforcement of intermediate grade steel. Plain columns 8¼-in. diameter.

Col. No.	Designed Concrete Str. lb. per sq. in.	Percentage of Reinforcement		Column Str. Fast Loading		Cyl. Str. lb. per sq. in.	Column Str. Slow Loading		Cyl. Str. lb. per sq. in.
		Long.	Spiral	Total lb.	Unit lb. per sq. in.		Total lb.	Unit lb. per sq. in.	
Intermediate Grade Vertical Bars									
a	2000	3.98	1.24	269,000	5350	2950	252,000	5000	2180
b	2000	3.98	1.24	267,500	5310	2490	240,000	4770	2100
c							262,800	5210	2230
d							269,000	5340	2135
Average				268,250	5330	2720	255,950	5080	2160
a	3500	3.98	1.24	276,000	5490*	3530	310,000	6150	3910
b	3500	3.98	1.24	298,000	5920	3770	271,000	5380*	3790
Average				287,000	5705	3650	290,500	5765	3850
a	5000	3.98	1.24	362000,	7200	5550	383,000	7600	4880
b	5000	3.98	1.24	360,000	7160	5710	381,000	7560	5530
Average				361,000	7180	5630	382,000	7580	5205
Structural Grade Vertical Bars									
a	2000	3.98	1.24	217,000	4310	2310	231,000	4590	2150
b	2000	3.98	1.24	226,000	4490	2050	227,000	4510	2060
Average				221,500	4400	2180	229,000	4550	2105
a	3500	3.98	1.24	265,000	5270	3650	284,000	5650	3470
b	3500	3.98	1.24	271,000	5390	3450	300,000	5950	3590
Average				268,000	5330	3550	292,000	5800	3530
a	5000	3.98	1.24	352,000	7000	6080	360,000	7150	5640
b	5000	3.98	1.24	354,000	7040	5620	336,000	6670	5800
Average				353,000	7020	5850	348,000	6910	5720
Rail Steel Vertical Bars									
a	2000	3.98	1.24	288,000	5730	2250	267,000	5300	2070
b	2000	3.98	1.24	294,000	5850	2330	285,500	5680	2170
Average				291,000	5790	2290	276,250	5490	2120
a	3500	3.98	1.24	340,000	6760	3550	350,000	6950	3770
b	3500	3.98	1.24	347,800	6900	3450	330,200	6570	3570
Average				343,900	6830	3500	340,100	6760	3670
a	5000	3.98	1.24	456,000	9080	5510	414,500	8240	5500
b	5000	3.98	1.24	378,500	7540	5200	433,800	8600	5350
Average				417,250	8310	5355	424,150	8420	5425

*These columns showed large deflections at failure.

TABLE 10—RESULTS OF TESTS OF SERIES 2—CONTINUED

All reinforced columns 8 in. core diameter, 5 ft. long, 2 companion columns of each type. Columns cured 56 days in moist room and tested moist. All spiral reinforcement of intermediate grade steel. Plain columns 8¼-in. diameter.

Col. No.	Designed Concrete Str. lb. per sq. in.	Percentage of Reinforcement		Column Str. Fast Loading		Cyl. Str. lb. per sq. in.	Column Str. Slow Loading		Cyl. Str. lb. per sq. in.
		Long.	Spiral	Total lb.	Unit lb. per sq. in.		Total lb.	Unit lb. per sq. in.	
				Intermediate Grade Vertical Bars					
a	3500	0	1.24	209,000	4150	3560	201,500	4000	3580
b	3500	0	1.24	208,000	4130	3660	212,600	4220	3490
Average				208,500	4140	3610	207,050	4110	3535
a	3500	1.57	1.24	226,000	4500	3440	234,000	4650	3870
b	3500	1.57	1.24	241,500	4800	3290	243,500	4840	4050
Average				233,700	4650	3365	238,750	4745	3960
a	3500	5.94	1.24	314,000	6240	3660	323,000	6420	3190
b	3500	5.94	1.24	337,000	6700	3280	327,200	6500	3630
Average				325,500	6470	3470	325,100	6460	3410
a	8000	0	1.24	390,000	7760	6920			
b	8000	0	1.24	421,000	8380	6960			
Average				405,500	8070	6940			
a	8000	1.57	1.24	400,000	7950	7570			
b	8000	1.57	1.24	420,200	8350	7310			
Average				410,100	8150	7440			
a	8000	3.98	1.24	463,000	9200	7690			
b	8000	3.98	1.24	482,000	9600	6790			
Average				472,500	9400	7240			
a	8000	5.94	1.24	488,000	9710	7310			
b	8000	5.94	1.24	450,000	8950	7100			
Average				469,000	9330	7205			

Table 10 concluded next page.

TABLE 10—RESULTS OF TESTS OF SERIES 2—CONTINUED

All reinforced columns 8 in. core diameter, 5 ft. long, 2 companion columns of each type. Columns cured 56 days in moist room and tested moist. All spiral reinforcement of intermediate grade steel. Plain columns 8¼-in. diameter.

Col. No.	Designed Concrete Str. lb. per sq. in.	Percentage of Reinforcement		Column Str. Fast Loading		Cyl. Str. lb. per sq. in.	Column Str. Slow Loading		Cyl. Str. lb. per sq. in.
		Long.	Spiral	Total lb.	Unit lb. per sq.in.		Total lb.	Unit lb. per sq.in.	
				Plain Concrete					
a	2000	0	0	89,000	1665	2170	85,000	1590	2280
b	2000			100,000	1870	2125			
c	2000			100,000	1870	2505	103,000	1930	2240
d	2000			89,500	1675	2400			
Average				94,600	1770	2300	94,000	1760	2260
a	3500	0	0	140,000	2620	4100			
e	3500			145,000	2720	3570	Broken		
f	3500			139,000	2600	3570	162,400	3040	4100
c	3500			160,000	3000	3740			
Average				146,000	2730	3745			
b	5000	0	0	260,300	4870	5850			
c	5000			246,000	4600	5010	253,600	4740	5520
e	5000			268,100	5020	5435			
d	5000			240,000	4500	....	236,000	4420	5450
Average				253,600	4760	5430	244,800	4580	5485
a	8000	0	0	333,000	6230	6820			
b	8000			336,000	6290	6840	302,000	5650	8165
f	8000			282,500	5470	7480			
g	8000			335,000	6260	7720	276,000	5160	7015
Average				324,100	6060	7215	289,000	5400	7590

Acknowledgment—The investigation has required the help of a considerable laboratory staff. Especial acknowledgment is made to R. L. Brown, Research Associate, and to L. R. Tucker and T. G. Taylor, Research Graduate Assistants, who have given valuable assistance in the testing program, and in the reduction of the data.

ABSTRACTS

ABSTRACTS

of the current literature of cement and concrete

PREPARED UNDER THE DIRECTION OF THE
AMERICAN CONCRETE INSTITUTE
COMMITTEE ON ABSTRACTS

J. C. WITT, Chairman, (*Materials*)
H. F. GONNERMAN (*Properties of Concrete*)
F. E. RICHART (*Engineering Design*)
REXFORD NEWCOMB (*Architectural Design*)
THEODORE CRANE (*Field Construction*)
W. D. M. ALLAN (*Shop Manufacture*)

EDITED BY DWIGHT F. JENNINGS

ABSTRACTORS

JOHN E. ADAMS
F. O. ANDEREGG
ROBERT W. ABBETT
ODD ALBERT
A. G. ALEXANDER
S. O. ASPLUND
C. BACHMANN
P. H. BATES
A. A. BRIELMAIER
A. E. BEITLICH
MILES N. CLAIR
C. G. CLAIR
IRA L. COLLIER
M. A. CORBIN
R. J. DIETRICH
E. L. ERIKSEN
H. FRAUENFELDER
W. I. FREEL
P. J. FREEMAN
H. J. GILKEY
A. T. GOLDBECK
J. E. GRAY
E. S. HANSON
S. C. HOLLISTER
A. J. HOSKIN
F. H. JACKSON
V. P. JENSEN
D. E. LARSON
H. WALTER LEAVITT
L. G. LENHARDT
A. E. LINDAU
ARTHUR R. LORD
H. J. LOVE
P. MCKIM
JOSEPH MARIN
N. D. MITCHELL
REXFORD NEWCOMB
J. C. PEARSON
C. E. PROUDLEY
F. E. RICHART
N. H. ROY
J. H. SHANK
EDMUND SHAW
A. SIZE
T. U. TAYLOR
T. THORVALDSON
G. E. TROXELL
H. S. VAN SCOYOC
STANTON WALKER
G. M. WILLIAMS
C. R. YOUNG
ROY N. YOUNG

MATERIALS

AGGREGATES

Sand and gravel. "Sand und Kies." E. TRUEMPENER. 1930, Kalkverlag, Berlin (Germany), R. M. 4.20. Reviewed in *Zement* (Germany), Nov. 27, 1930, V. 19, No. 48, p. 1146.—Book deals with application of sand and gravel in concrete and other industries and describes properties, which are necessary for such purposes. Modern practice demands higher standards of purity and uniformity. A comprehensive list of references and many illustrations are given.—A. E. BEITLICH

The sand and gravel industry and its problems—Part II. JOHN ZOLLINGER. *Rock Products*, Dec. 20, 1930, V. 33, No. 26, p. 56.—Dredging pump described has special protection by pressure water which protects both runner and shell. This has decreased repair bill two-thirds, based on two years experience. Dredge with portable hull and an 8-in. pump—has hull made of four 8 x 16-ft. pontoons. Four men can easily handle and unload one pontoon. The cabin is in ten sections. The advantages of portable type are lower expenses of transportation and launching and a greater resale value.—EDMUND SHAW

Glacial sand-gravel aggregate found suitable for paving work. L. H. KOENITZER. *Concrete*, Jan., 1931, V. 38, No. 1, p. 29.—Glacial material available in eastern Nebraska and portions of Kansas has maximum size of 1½ in. Most material passes ¾-in. mesh sieve and has fineness modulus of 4.10 to 4.50. Study was made on over 450 9-in. spheres for wear tests, 140 6 by 6 by 35-in. beams for flexure tests, and 500 3 by 6-in. cylinders for compression tests, to determine range of possibilities of the aggregate. Data show that for variable mixes, with constant water-cement ratios and fineness moduli of 3.50 and 4.50, mixes containing 20 per cent passing 28-mesh sieve gave higher strength values than similar mixes containing 10 and 15 per cent passing same sieve; increase in water-cement ratio decreased strength and increased per cent of wear. Mixes giving greatest strength are listed. Total aggregate on six miles of paving showed average modulus of rupture greater than 560 lb. per sq. in. at 10 days; compressive strength of 3800 lb. per sq. in. at 28 days. Total aggregate satisfactory when precautions are taken for expanding and contracting of concrete.—C. BACHMANN

New England quarry plant makes periodical changes. *Rock Products*, Nov. 22, 1930, V. 33, No. 24, p. 41-46.—At beginning of season just passed, the Middlefield, Conn., plant of the Connecticut Quarries Co. changed from steam to electric power and from rotary to vibrating screens beside making other improvements reviewed. Power change was made by installing a single 500-h. p. synchronous motor to drive principal crushers and screens. High and uniform power factor resulting has caused a material saving in power cost. Vibrating screens have shown lower power and general maintenance costs and a higher cost for screen fabric, but are found on the whole to be more economical to operate than rotary screens. Quarry face of hard trap rock is 1,500 ft. long and from 80 ft. to 190 ft. high. It is worked by a combination of coyote holes and snake holes, each shot being carefully designed on the drawing board. The maximum burden allowed is 60 ft. Loads as great as 54,000 lb. of 40 per cent powder have been shot. The recovery is estimated to run between 5 and 5½ tons of rock per lb. of powder and practically no secondary shooting is required. Haulage is by motor truck.—EDMUND SHAW

A year's operating experience with a new crushing plant. NELSON SEVERINGHAUS. *Rock Products*, Sept. 27, 1930, V. 33, No. 20, p. 33-38.—The superintendent of Consolidated Quarries Corp., Lithonia, Ga., tells what a year's experience with a new plant has taught. Blast hole drilling originally

was 18 ft. spacing with 20 ft. burden. The rock broke clean from the face but fragmentation was poor requiring too much expensive block-holing. The burden was reduced to 16 ft. for the second shot with much better fragmentation. Drilling is now being tried with 5⅝-in. holes set 18 x 20 ft. The charge will build up faster in this size hole and less stemming will be required. It cost 79¢ to break the average oversize piece by mudcapping and 34¢ by blockholing. As the quarry is in a locality where the same rock is cut for building stone and paving blocks, the method of breaking by short holes and wedges was tried. Cost was about same as for blockholing but larger pieces were left to be handled by shovel. Reversable control on crusher motor has saved cleaning out crusher after shut down. Author believes lowering crusher speed from 170 to 125 r. p. m. would reduce peak load, lessen wear on bearings and still give sufficient capacity. Due to accumulation of fines in bins and stockpiles, recleaning vibrating screens have been put in over loading bins. Some washed sand is being produced from screenings by the use of an Anaconda type rising current classifier.—EDMUND SHAW

Effect of flat and elongated pieces in crushed stone used as an aggregate in concrete pavements. A. T. GOLDBECK. *Crushed Stone J.*, May, 1930, V. 6, No. 5, p. 2-11.—This investigation was designed to answer two questions: (1) Are flat pieces actually harmful in finishing of concrete pavements? (2) What effect do flat pieces have on strength of concrete? Flat fragments were defined as those whose length exceeds five times the least average dimension. Only larger size flat pieces, those retained on a 1¼-in. screen, were considered harmful. Slabs 3 ft. wide, 6 ft. long and 6 in. thick were made of 1:2:3½ concrete with 0, 5, 10, and 15 per cent of flat pieces, and of 1:2:4 concrete with 0, 5, and 10 per cent of flat pieces. Two slabs of 1:2:4 concrete were made in which gradation of stone was varied. Best index of difficulty of finishing was considered appearance of concrete after each successive passage of steel screed, and finally, after use of wood float. Slabs were broken into beams which were tested at age of 28 days for modulus of rupture. Following conclusions were drawn: (1) Presence of up to 10 per cent of flat pieces in stone has no noticeable effect on difficulty of finishing 1:2:4 concrete, provided same stone with flat pieces removed produces concrete which may be finished without difficulty. (2) The presence of flat pieces only slightly increases percentage of voids in stone and requires no change in water-ratio to produce concrete of same consistency as obtained with stone having zero per cent of flat pieces. (3) Changes in gradation of stone have a far greater effect on percentage of voids and mortar voids ratio than flat pieces. (4) The 1:2:3½ mixes were more workable than 1:2:4 mixes. They have much greater factor of safety against variables which may occur in aggregates than 1:2:4 mixes. (5) Flat pieces up to 10 per cent in 1:2:4 mixes and 15 per cent in 1:2:3½ mixes do not decrease strength of concrete. (6) Indications are that flat pieces do not lie in a position such as to cause trouble at surface of a concrete slab.—J. E. GRAY

CEMENT

Investigation of portland cements, rich in ferric oxide—II and III. SHOICHIRO NAGAI AND K. AKIYAMA. *Kogyo Kwagaku Zasshi, J. of Soc. Chem. Ind.* (Japan), 1930, V. 33, p. 286-90, 421-8. Reviewed in *Zement* (Germany), Nov. 13, 1930, V. 19, No. 46, p. 1096.—Cements were burned between 2400 and 2460° F. with a ferric oxide content of 7½ to 11 per cent. Raw materials were limestone, puzzolana, copper slag and clay. Mortar strength (1:3) was tested by means of small-piece testing method, invented by author. Compressive strength is lowered when ratio $CaO:SiO_2$ is decreased.—A. E. BEITLICH

Movements of materials in rotary kiln and power consumption. A. NILSSON. *Zement* (Germany), Nov. 20, 1930, V. 19, No. 47, p. 1105.—Mathematical study of movement of materials in rotary kilns. Power consumption

can be calculated by means of formula derived by author. Factors taken into consideration are: weight of materials in kiln, number of revolutions of kiln, changes in weight of mix due to calcination and drying and internal friction of materials. Formula is not considered for use in mill practice.—A. E. BEITLICH

Dusting of portland cement clinker. KATSUZO KOYANAGA. *Rock Products*, Nov. 22, 1930, V. 33, No. 24, p. 66-68.—The opinion of Dyckerhoff is quoted, that dusting of clinker comes with a change of dicalcium silicate from the β to the γ modification which is accompanied with about 10 per cent increase in volume. The author then describes his investigation of dusting clinker which was made through the accidental inclusion of a piece of flint stone, probably, with a raw meal of known composition. Microscopic examination showed the original clinker to be largely Alit. The dusting portion of clinker was mainly Belit. It was much higher in silica and lower in lime than original clinker. Tests were made by fusing pure reagents to $CaO.SiO_2$, 2 $CaO.SiO_2$, and 3 $CaO.SiO_2$. The melt of 2 $CaO.SiO_2$ broke into white powder. Tests were then made with two kinds of raw meal, one high and the other low in sesquioxide. Silica, some of which was taken from such bricks as are used in kiln linings, was added and rods were molded which were melted in an oxy-acetylene flame, allowed to drop in a heated crucible and then cooled. Six series of these melted clinkers were plotted on a ternary diagram. The line connecting the dusting points was nearly straight through the points of 2 $CaO.SiO_2$ mixtures. When the dusting clinkers were tabulated dusting was shown to occur through a wide range of chemical composition. But the contents of lime and silica can never be higher than 64.92 per cent CaO or 35.08 per cent SiO_2. The higher the content of sesquioxide lower the content of lime and silica and the longer time it will take to begin dusting. It is thought that dusting more often occurs with silica than aluminous brick lining.—EDMUND SHAW

Investigations on operation and efficiency of air separators. II. P. ROSIN AND E. RAMMLER. *Zement* (Germany), Oct. 16, 23, 30, Nov. 6, 1930, V. 19, No. 42, 43, 44 and 45, p. 984-8, 1011-4, 1035-8, 1060-1.—An illustrative case is worked to analyze process of grinding in tube mill with an air separator. Tests were conducted with coal. Amount of feed was varied while all other conditions were kept constant. Samples were taken at various places and their fineness determined. Influence of rate of feed and number of revolutions of air separator fan on (1) fineness of end product, (2) number of circulations of feed, (3) load of elevator and actual load of mill, (4) efficiency of air separator and (5) power consumption were carefully examined. Results of tests were used for expression of encountered conditions by mathematical formulas. Use of these formulas and their application to mill practice are especially emphasized. Reference is made to work by Rosin and Rammler, *Zement* (Germany), June 27, July 18, Aug. 1 and 8, 1929, V. 18, No. 26, 29, 31 and 32; A. B. Helbig, *Ibid*, Jan. 9, 1930, V. 19, No. 2, p. 25-8, *J. Am. Concr. Inst.*, April, 1930, V. 1, No. 6, Abstr. Sect. p. 69.—A. E. BEITLICH

Reply to the article by Alton J. Blank on ignition losses. KATSUZO KOYANAGI. *Rock Products*, Dec. 20, 1930, V. 33, No. 26, p. 67.—Discussing Mr. BLANK's view that the harder clinker is burned the less moisture it will absorb, the author says moisture absorbed in storage is function of two factors, (1) quantity of moisture in atmosphere and (2) hardness of clinker. But a higher burned clinker may absorb more moisture than a soft burned clinker if it is stored in atmosphere with more moisture. Author's experience is quite opposite to Blank's statement that he has never known strength qualities of cement to be bettered by storage whether in humid or dry atmosphere. Hourly samples of cement are taken at outlet of mills and packers and mean strength found. Mean strength after one month of storage is always a little higher than that of same fresh ground cement. Many tests on effects of storage have shown

that storage of moderate length is effective in increasing strength of cement, or at least it is not harmful.—EDMUND SHAW

Investigation of portland cement and its constituents by means of measurements of vapor pressure. F. KRAUSS AND G. JOERNS. *Zement* (Germany), Nov. 6, 1930, V. 19, No. 45, p. 1054-5.—New method is proposed, which enables investigator to study formation of hydrates and to observe kind of hydrates which are present. Nature of process (if it is formation of crystals or colloids) and amount of water which is taken up can be studied. Two different ways are worked out, one is determination under isothermal conditions and one under isobar conditions. Latter way is superior for study of portland cement. Substance is exposed to vacuum at certain temperatures. Vapor pressure will always be constant when true hydrate is present until this is decomposed, which fact will set off state of equilibrium. Micro-tension-eudiometer by Krauss and Schriever and its operation is described.—A. E. BEITLICH

Reaction of carbon dioxide and acetic acid on portland cement. HANS TH. BUCHERER AND F. W. MEIER. *Zement* (Germany), Nov. 27, 1930, V. 19, No. 48, p. 1134-8.—Suspensions of cement in water were treated with carbon dioxide and amounts of lime and silica, which went into solution, were determined. Intention of investigators was to remove lime as completely as possible without attacking silica, but it was found that besides calcium bicarbonate considerable silica was dissolved. Acetic acid was then used in 1 per cent solution in addition to carbon dioxide with specially prepared indicator. Carbon dioxide was introduced under constant stirring and calculated amount of 1 per cent acetic acid was added drop by drop avoiding any excess acid. About 24 per cent of total silica was dissolved. Further tests were carried out with acetic acid alone and phenolphthalein as indicator. Results of tests are plotted in curves and given in tables. Comparisons were made between reactions of great number of different cements (portland and high alumina) with acetic acid and their strength properties. It was found that cements with low compressive strength show steep titration curves with only small consumption of acetic acid while cement with similar chemical composition but high compressive strength consume more acid during titration and have flat curves.—A. E. BEITLICH

Researches on the rotary kiln in cement manufacture. Part VI. GEOFFREY MARTIN. *Rock Products*, Oct. 25, 1930, V. 33, No. 22, p. 43-44.—Heat radiated from kiln walls in decarbonating zone is 43,338 S B. t. u. in 1 hr., where S equals the surface in feet. To make 1 lb. of clinker, decomposing 1.1905 lb. $CaCO_3$ at 805° C., 811.92 B. t. u. are required. Heating 1-lb. material from 805° C. to 100° C. requires 148.7 B. t. u. The total heat per lb. of clinker which is required in the decarbonating zone is therefore 148.7 plus 811.9 = 960.6 B. t. u. And for one ton of 2240 lb. it is 2.152×10^6 B. t. u. The coal required is shown to be $2240 \frac{M}{y}$ lb. per hr. which evolves $28.23 \times 10^6 \frac{M}{y}$ B. t. u. per hr. Unbalanced residue of heat must be counterbalanced by same amount of heat radiated from kiln walls, because the temperature of walls must steadily increase as heat plays upon them until they radiate back as much heat as they receive. Equating the heat radiated away against the free heat from the coal gives: $43358\ S = M \times 10^7 \left(\frac{28.23}{y} - 2.152\right)$ which calculates to $S = M \left(\frac{651}{y} - 49.63\right)$ If y is assumed to be 3 then S equals 167.4 M. A table is appended giving the surface in the decarbonating zone required for outputs of 1 to 20 tons per hr., based on this value of y. This formula is said to be new. The length of the decarbonating zone is figured from

surface and diameter. It has been previously shown that $d = 2.98\sqrt{M}$ and as $S = \pi\, dL$ then $L = 17.88\sqrt{M}$. A table gives the required lengths of decarbonating zones for outputs of 1 to 45 tons per hr. and diameters from 3 ft. to 20 ft.—EDMUND SHAW

Researches on the rotary kiln in cement manufacture. Part VII. GEOFFREY MARTIN. *Rock Products*, Nov. 8, 1930, V. 33, No. 23, p. 36.—Theoretical deductions of sizes of preheating and drying zones of kilns are difficult. Empirical experiment has shown following formulas to give approximately correct results for wet process kilns, slurry to contain 40 per cent moisture, where L is the length of the kiln: Cooler end, $2/21\ L$; Clinkering zone, $4/21\ L$; Expulsion zone, $5/21\ L$; Preheating zone, $3/21\ L$; Drying zone, $7/21\ L$. The relation of length L to diameter d is worked out to: $L = 25\ (d - 1.5) + 0.2\ (d - 1.5)^3$. For dry process kilns the relation is: $L = 20\ (d - 1.5) + 0.2\ (d - 1.5)^3$. d is stated to be diameter of iron shell in the decarbonating and drying zones. Where diameters vary up and down the kiln d must be taken as the average diameter. Commenting on this article, the Editor, *Rock Products* says: "Apparently about 260 to 365 ft. is the proper length for the usual American wet process kiln, 11 ft. 3 in. diameter of the steel shell. For the dry process the kiln should be only about 50 ft. shorter, which would seem to prove most of our dry kilns too short."—EDMUND SHAW

Researches on the rotary kiln in cement manufacture. Part VIII. GEOFFREY MARTIN. *Rock Products*, Nov. 22, 1930, V. 33, No. 24, p. 47-49.—Theory of expulsion of CO_2 from lime and magnesian carbonates is discussed with an explanation of disassociation pressures and Johnstone's table. It is concluded that in a rotary cement kiln CO_2 begins to evolve at 805° C. at the point where the raw material enters the calcining zone. But as it enters the sintering zone the temperature at which it begins to evolve CO_2 is reduced to 748° C. Practically this means that evolution of CO_2 is almost infinitely slow at these temperatures at the points in the kiln mentioned but as the temperature of the raw material increases beyond these limits the rapidity of expulsion rapidly increases. At 900° C. the bulk of the CO_2 has probably been expelled, last traces going off probably at 1100° C. The time factor is of great importance. Years would be required at 805° C. at start of calcining zone whereas a few minutes would suffice if raw material temperature were brought to 900° C. It would be very desirable to determine minimum time of passage through furnace at temperatures from 800° C. to 1100° C. to arrive at time constant for different temperatures. Influence of speed of kiln gases over surface of material should also be investigated. Having collected such data one would be in a position to design a kiln on scientific lines. At present kilns are designed on rules founded on experience for lack of chemical and physical data. An appendix to this part gives a digest of references to disassociation of carbonates published in literature relating to cement.—EDMUND SHAW

Indirect determination of lime in cement and similar materials with standard sodium oxalate (Sorensen). A. HEISER. *Zement* (Germany), Dec. 4, 1930, V. 19, No. 49, p. 1154-5.—Titrometric method for determination of lime is described. Latter is precipitated with a known excess of sodium oxalate which in turn is back-titrated with solution of potassium permanganate of known concentration. Sodium oxalate of highest purity is used for this purpose. Potassium permanganate solution is boiled and filtered before standardization. Procedure of determination is as follows: Filtrate from R_2O_3 precipitate (of 1.0000 g. sample) is heated near boiling temperature, made acid with strong HCl and then 1.9832 g. sodium oxalate is added. After boiling for about 5 minutes more HCl is added until precipitate is dissolved. Calcium oxalate is then precipitated by adding slowly strong ammonia to slight excess. After settling, solution is filtered into 500 cc. measuring flask and carefully washed with hot water. After cooling flask is filled with water

to mark and 100 cc. are withdrawn with a pipette. To this latter portion are added 400 cc. water, 20 cc. strong sulphuric acid and 20 cc. manganese sulphate solution. After heating to 158° F. it is titrated with 1/10 normal potassium permanganate solution. Volume corrections are necessary when temperature in laboratory changes frequently. Method is simple and gives good results.—A. E. BEITLICH

Marquette has new tow boat. *Rock Products*, Dec. 6, 1930, V. 33, No. 5, p. 68.—The "William Dickinson," 125 ft. long, 26-ft. beam and 7-ft. molded depth, new tow boat of Marquette Cement Manufacturing Co., tows four self-unloading barges, each of 5000 bbl. capacity, with Fuller-Kinyon pumps for pumping cement ashore. It is powered with two Atlas-Diesel engines of 350 h. p. at 225 r. p. m., connected to propellers by air operated clutches that slip with an overload. A new hydro-pneumatic control of clutches has been designed by the builders. The propellers, 6 ft. dia., 63-in. pitch, are solid semi-steel. Double cast iron rudders are used foreward and aft of each propeller. A hydro-pneumatic steering gear operates either set independently. There is no wheel, steering being by levers which are synchronized with helm angles. Hull is all steel. Stern is straight sided and flat topped to faciliate the easy entrance and discharge of water to the propellers.—EDMUND SHAW

Seven 120 ft. cement kilns replaced by one 305 ft. in length. *Pit and Quarry*, Dec. 17, 1930, V. 21, No. 6, p. 25-28.—Plant of the Wolverine Portland Cement Co., at Quincy, Mich., in operation since 1899 has been altered numerous times and, with the changes made in 1930, is up to date. Raw material is marl, excavated by a clamshell-bucket dredge and pumped into large wooden barges which are towed from the small chain of lakes, through a barge canal, to the dock at the main plant. Dredging is precluded during about three months each winter. The marl is pumped from barges, by a return-air system, into steel storage tanks of a combined capacity of 540 cu. yd. Samples are taken and clay is added as marl flows to pug-mill. From pug-mill the mixture goes first to a vat and thence to tube-mills. Slurry then flows to another vat and thence to tanks of 90 cu. yd. each. A long screw conveyor takes slurry to the kiln building where it is elevated and fed to kiln which is 305 ft. long by 10 ft. in diam. and is driven by a 100-h. p. motor at one revolution in 58 sec. Gases from kiln are withdrawn by a 10-ft. fan. Coal is dried, pulverized and pumped pneumatically into steel bin whence it is fed to the air stream furnished by a 4-ft. fan, into the firing end of the kiln. Electric recording pyrometer registers the temperature of the exhaust gases. Clinker drops into a pit and thence by two drag-chain conveyors to a pan conveyor which leads to storage. Drag-chains can move the clinker to a cooler which discharges to storage. A monorail hoist with a 1-cu. yd. clamshell bucket reclaims the clinker and discharges it to a scale where gypsum is added. The material is then elevated to feed nine 30-in. mills. Final grinding is in two tube-mills. Cement goes to a 14-ft. air separator and the coarse fraction returns to tube-mills. Finished product is taken by screw conveyors and elevators to a bin storage.—A. J. HOSKIN

Strength properties of cements during 1930. (See PROPERTIES OF CONCRETE.)

MISCELLANEOUS

Some of the problems of coloring concrete. GEORGE RICE. *Concrete Products*, Dec., 1930, V. 39, No. 6, p. 16-17.—This article takes up the coloring of concrete from a scientific standpoint, showing the molecular structure of color used in cement and the manner in which it may be attacked by color-weakening bacilli. The author also discusses methods for preventing the weakening of colors, as well as the interaction of one color upon another.—E. S. HANSON

Studies on fundamental synthesis of calcium aluminates and their hydration—Part 7. SHOICHIRO NAGAI AND RYUICHI NAITO. *Kogyo Kwagaku Zasshi, J. Soc. Chem. Ind.* (Japan), 1930, V. 33, Supplemental Binding, No. 8, p. 315-8B. Reviewed in *Zement* (Germany), Dec. 11, 1930, V. 19, No. 50, p. 1192.—Heat tests with Jaeneckeit showed that this compound consists at 2642° F. of 1 tricalcium aluminate, 2 dicalcium silicate and 1 CaO. Between 2732° and 2822° F. combination of CaO and dicalcium silicate to tricalcium silicate takes place. Eutectic mixtures of β-$2CaO.SiO_2$ and $5CaO.3Al_2O_3$ were obtained by heating to 2462° F. This mixture has relatively high strength properties. Similar tests were conducted with gehlenit.—A. E. BEITLICH

Hydrates of calcium sulfate. F. KRAUSS AND G. JÖRNS. *Tonind. Ztg.* (Germany), Nov. 24 and 27, 1930, V. 54, No. 94, p. 1467-8, No. 95, p. 1483-4.—The great variability of results reported in literature for hydrates of calcium sulfate occurs because of sensitiveness of these compounds to changes in grain size and to experimental conditions. Previous history is also important. The work of Linck and Jung establishing the zeolitic character of the hemihydrate is confirmed. To prepare pure hemihydrate start with freshly precipitated dihydrate, heat for a short time at its decomposition temperature, cool quickly where two forms are in equilibrium to reduce the production of anhydrite. The transformation took place at 59° C. and at a vapor pressure of 7 mm. in the absence of air.— F. O. ANDEREGG

Action of gypsum, swelling and hydration processes. F. TIPPMANN. *Zement* (Germany), Oct. 30, Nov. 6, 13, 20, 1930, V. 19, No. 44-47, p. 1030-5, 1056-60, 1080-6, 1106-12.—Opinions about effect of gypsum and anhydrite on setting properties of cement differ widely. Berger, Meade and Budnikoff have in general no objection to use of anhydrite as portland cement retarder, while Platzmann and Hart do not favor its use. Comprehensive tests were made to clear up this problem. Four different retarders: (1) raw gypsum rock of high purity, (2) natural pure anhydrite, (3) semihydrate ($CaSO_4.1/2H_2O$), produced from gypsum rock by heating for several hours at 311° F. and (4) stucco, regenerated dihydrate from waste of clay industry. Grinding tests were carried out with a well burned clinker in great laboratory ball mill in lots of about 70 lbs. over 5 hour periods. Additions of retarding agents ranged from 1 to 6 per cent. Obtained products were tested for fineness, setting time, 3-, 7-, 28-day compressive and tensile strength after combination storage, volume constancy, Le Chatelier test and SO_3 content. Additional tests were made with 2.5 per cent raw gypsum rock and 4.5 per cent anhydrite for 16 hours. Temperatures in mill during grinding process were carefully determined. It was found that anhydrite shows less retarding effect on clinker than the other modifications with water of crystallization. Slow setting products were prepared with 2 per cent raw gypsum, 1 per cent semihydrate, 2 per cent stucco and 4 to 5 per cent anhydrite. Constancy of volume was not affected by these high additions. Changes in SO_3 content of samples seem to indicate that a partial thermal dissociation takes place, caused by heat in mill. Regenerated dihydrate is not affected and needs higher temperatures. Author discusses opinions of numerous investigators (Biehl, Platzmann, Killig, Kuhl, Probst, Dorsch, Schmidt and Haehnle) and compares their findings with his own results. He claims that no calcium sulfoaluminate is formed when cement and water react together. This compound was prepared synthetically by treating white portland cement of the Medusa Portland Cement Co. (Ohio), with saturated solution of gypsum. Reactions of calcium sulfoaluminate and semihydrate with water and progress of crystallization with gypsum solutions of different concentrations were studied under microscope. Investigation includes studies of formation of colloidal gels in gypsum solution in connection with swelling tests. Special built apparatus allows microscopic observation of a stirred mixture of cement or cement compounds in gypsum solutions or lime water. Two phases of reaction could be observed: (1) electrolyte causes agglomeration of fine

particles without destroying their structure (apparent swelling process) and (2) true swelling process of solid residue and formation of new substances (silicate needles and aluminate plates) takes place; gypsum has little influence on this phase of reaction, while concentration of calcium hydroxide has great effect. Retarding effect of gypsum seems to be due to fact that a weak gypsum solution retards speed of solution of lime causing slower formation of hydration products. Silicate needles and an abundance of a transparant gelatinous substance which contains silica and lime (probably a pseudo-crystalline or liquid-crystalline modification of a calcium silicate) were found between set cement plates under water storage. Transformation of calcium hydroxide crystals from a colloidal to a crystalline form could be observed under microscope. Synthetic calcium aluminate hydrates were prepared and their optical properties determined. Finally solution phenomena of cement in sugar solutions of different concentrations and formation of crystal phases were examined. Thirty-nine references, 28 microphotographs, 11 graphs and several illustrations are given.—A. E. BEITLICH

PROPERTIES OF CONCRETE

The chemistry of the resistance of concrete to corrosion. WILLI SERKIN. *Betonwerk* (Germany), June 15, 1930, V. 18, No. 24, p. 334.—A discussion of action of soluble portion of cement in connection with insoluble aggregates, and of the influence of water on lime content of cement. Methods now in vogue to protect concrete by integral methods, changing the soluble portions into insoluble, surface and impregnation materials are fully discussed and described. It is cited that as yet no universal method of protecting concrete has been discovered.—H. FRAUENFELDER

Crystal Springs dam in California, 40 years old concrete still watertight. *Zement* (Germany), Oct. 23, 1930, V. 19, No. 43, p. 1022.—Data of inspection of Crystal Springs dam, which is still in perfect condition after a 40 year period, indicate that this condition is due to an extraordinarily careful construction control, use of perfect aggregates and systematic treatment of finished concrete.—A. E. BEITLICH

Overheating of aggregates found detrimental to concrete. WILLIAM H. BACHELDER. *Eng. News Record*, Dec. 18, 1930, V. 105, No. 25, p. 973.—In cold weather it is often necessary to heat concrete aggregates. Some specifications require that concrete when placed shall have temperature of from 70 to 140° F. This temperature has some effect on chemical reaction that takes place when cement sets. It has been assumed that effect of higher temperatures on resulting concrete is beneficial. A series of tests in Minnesota highway department laboratory showed that temperatures above 70° F. should not be used if the full development of strength inherent to concrete is to be obtained. Concrete placed at 130° F. loses approximately 20 per cent of its strength.—D. E. LARSON

Artificial influencing of strength of concrete—Part 3. C. R. PLATZMANN. *Zement* (Germany), Nov. 27, 1930, V. 19, No. 48, p. 1139-41.—Several surface hardeners for concrete are discussed. Special consideration is given to "Metalicron," a product made by Master Builders in Cleveland, Ohio. Properties of this material were carefully studied and abrasion of concrete specimens treated in different manners were measured. Photographs show corrosive influence of lactic acid, sugar solutions and acids from fats and oils. Microphotograph of Metalicron is given. A surprisingly high increase in strength was found of specimens stored in a solution containing 8 per cent sodium sulphate and 2 per cent magnesium sulphate.—A. E. BEITLICH

Retempered mortar. *Concrete Constr. Eng.* (England), Dec., 1930, V. 25, No. 12, p. 696.—Results are given of tests made at the Building Research Station to determine effect on strength of mortar of remixing cement mortars up to periods of 30 to 45 minutes after mixing and after initial set has taken place. The rate of hardening of cement and consistency of mix are important factors in deciding final result. For slow hardening portland cement it may be possible to regauge cement mortar for periods up to possibly 4 hr. without loss in strength, while for rapid-hardening and aluminous cements there is risk of loss in strength occurring due to retempering. Two series of tests were carried out at Building Research Station. One series for normal slow hardening portland cement and the other for rapid hardening portland cement. Values are given of strengths for varying times of remixing. The general conclusion made was that the process of retempering cement mortar and concrete cannot be recommended.—JOSEPH MARIN

Control of concrete construction. "Die Baukontrolle in Betonbau." BREBERA. 1929, Eduard Gregrasin, Prague (Czecho-Slovak Republic). Reviewed in *Baulechnik* (Germany), Sept. 19, 1930, V. 8, No. 41, p. 636.—Tentative directions for production of concrete are derived under consideration of all experiences in recent years with big concrete construction jobs. After emphasizing necessity of construction control, author discusses factors which influence quality and strength of concrete. Ball-test by Heinzel for determination of setting time is described and also a method for testing binding properties of cement, found by determining flexural strength of 10-in. long mortar prisms. Special consideration is given to chapters on aggregates, methods for determination of impurities, practical gradations and proportioning according to Abrams fineness modulus. Mixing water and influence of natural water content of aggregates on strength are considered. Final chapter discusses equipment and apparatus for perfect concrete mixing.—A. E. BEITLICH

Mixing proportions not only factor affecting concrete strength. ALBERT MERCIOT. *Constructeur de Ciment Armé*, Oct., 1930, No. 133, p. 217-219.—Somewhat out-of-date to American readers are arguments collected here in favor of field control of concrete. Concrete strength should not be conditioned by mixing proportions, relative proportions of sand and gravel, quantity of mixing water, or method of working concrete. It should be sufficient to simply specify that after a certain lapse of time concrete on the job should show a resistance to rupture, which, in accordance with factor of safety desired, corresponds to working stresses assumed in the design. This will give great freedom to contractor in choice of his materials and methods. Uniformity of concrete will be secured on the job. Scientific control on the job will eliminate complaints of fraud. Already there exist in France companies which will give this type of service on reinforced concrete construction work.—M. A. CORBIN

Strength properties of cements during 1930. ALEXANDER HASCH AND LEOPOLD RISTER. *Osterr. Bauzeitung* (Austria), Nov. 15, 1930, V. 6, No. 46, p. 752.—Compressive and tensile strength data of cements tested during 1930 by Austrian Institute for Testing Materials in Vienna are tabulated. Such cements are ordinary portland cement, high early strength portland cement, Kuhl cement and several foreign brands of fused high alumina cements. Additional durability tests were carried out by storing specimens in various aggressive solutions—sulphuric acid (5 per cent), magnesium sulphate (5 per cent), ammonium nitrate (5 per cent), sodium nitrate (1 per cent) and saturated gypsum solution. Solutions were renewed every 2 weeks.—A. E. BEITLICH

Effect of shrinkage in large concrete dams. M. A. RENAUD. *Annales Ponts Chaussees* (France), March-April, 1930, V. 1, No. 2, p. 144-179.—Fissures produced by shrinkage of concrete in dams are important in their extent. Study of how fissures were produced can only be made after failure has taken

place, and then there is doubt as to what actually took place in the interior. Engineers in charge of the Saint-Marc dam observed concrete while dam was in different stages of completion and were thus able to note any fissures that formed and to remove them in time. Dam is of gravity type, 145 ft. high, and contains 97,500 cu. yd. of masonry. Method of construction is explained and stages in placing of concrete indicated by diagrams. Figures illustrate age of concrete of different parts of dam and proportions of aggregates. Extent of fissures was determined by means of colored water and detailed account is given of formation, growth and location of fissures. Extent of fissures is shown by means of diagrams of the cross-section, for different periods of time after the first evidence of a crack was noticed. Main deduction made from observations was that fissures were due to shrinkage of concrete, for construction was not advanced sufficiently for dead load of masonry to produce cracks, nor was temperature sufficiently low to cause fissures. Proposed measures to protect against formation of new fissures, and method of repairing dam in which fissures are already formed are outlined. Observations taken on Saint-Marc dam point out necessity of observing dam during its construction for fissures that might form. Fissures in another dam that has been in service for two years were found analogous to those in Saint-Marc dam.—JOSEPH MARIN

Valuation of shrinkage numbers of light-concrete. A. HUMMEL. *Zement* (Germany), Nov. 6, 1930, V. 19, No. 45, p. 1062-6.—Shrinkage of mortar and concrete is due to (1) shrinkage of cement, (2) shrinkage of aggregates, (3) gradation of aggregates and (4) amount of gauging water. Air moisture, temperature conditions and structure of concrete also affect shrinkage. Shrinkage can be reduced by use of lean mixtures with little gauging water; shrinkage process can be retarded by keeping concrete wet until a considerable hardness has been produced. For light-weight concrete, aggregates must have small coefficient of contraction besides heat insulating properties and light weight per cu. ft. Frequently shrinkage numbers of light concrete can be found which are much higher than those of normal concrete. A formula for comparison of shrinkage numbers is derived based on tensile strength, specific shrinkage and modulus of elasticity. Several different concretes are investigated from this point of view.—A. E. BEITLICH

Glacial sand-gravel aggregate found suitable for paving work. (See MATERIALS—AGGREGATES.)

Action of gypsum, swelling and hydration process. (See MATERIALS—MISCELLANEOUS.)

Advantage and versatility of cast stone. (See ARCHITECTURAL DESIGN.)

ENGINEERING DESIGN

BRIDGES

A method of arch design. G. P. MANNING. *Concrete Constr. Eng.* (England), Dec., 1930, V. 25, No. 12, p. 687-689.—Paper deals with "combined moment envelopes," diagrams representing total positive and negative moments for points along the arch. Moments plotted include those due to dead load, live load, crown drop, and temperature. The diagram given is the "envelope" for a specific case. The arch treated is as follows: span, 150 ft., rise 30 ft., open spandrel type with floor and roadway weighing 250 lb. per sq. ft., ribs spaced at 10 ft. centers, live load consists of one standard train including 50 per cent impact, E_c = 2,000,000, I_c = 545,000 in.^{4}, cross section of the rib at the crown 30 in. wide by 48 in. deep with 1.5 per cent steel top and bottom, and a temperature range of + or — 30 degrees. The moments were calculated for two types of profiles and comparison made.—JOSEPH MARIN

BUILDINGS

Large building structure with shock absorption. *Osterr. Bauzeitung* (Austria), Nov. 22, 1930, V. 6, No. 47, p. 767.—New construction principle was used for construction of "Rhenania-Ossag"-house in Berlin (Germany). No direct connection exists between inner structure and outer walls which fact makes this office building shock proof and reduces considerably, vibrations caused by street traffic.—A. E. BEITLICH

Reinforced concrete skeleton structure for an apartment and office building. JOS. HOLZAPFEL. *Zement* (Germany), Nov. 20, 1930, V. 19, No. 47, p. 1114-7.—Design and construction of building at street corner in Munich (Germany) was extremely difficult on account of very unfavorable underground and narrow site of construction. The building is 93.5 ft. long but only 16.4 ft. wide. Construction as reinforced concrete skeleton structure was only possible solution. Details are given of dimensions and reinforcements of structural parts.—A. E. BEITLICH

Cleveland railroads dedicate Union Terminal. H. D. JOUETT **and Foundations for the Cleveland Union Terminal.** C. P. MARSH. *Civil Engineering*, Nov., 1930, V. 1, No. 2, p. 77-88.—This recently completed $88,000,000 project included a 52-story tower supported on foundations extending 204 ft. below rail level. Quantities included 500,000 cu. yd. of concrete and 100,000 tons of steel work. The terminal serves the New York Central, Big Four, and Nickel Plate railroads. Preliminary foundation investigations involved much work of a pioneer nature including library studies, 8,000 linear ft. of test holes, soil analyses, and bearing tests on soil and rock samples and field tests of soil in place. This soil was fine sand containing about 13 per cent clay and 3 tons per sq. ft. was the final pressure allowed for retaining walls and spread footings placed upon it. At 60 or 70 ft. below basements there is hardpan of compact clay mixed with gravel on which a pressure or 7½ tons per sq. ft. was attained. The tower is supported on 16 concrete piers carried to rock for which 100 tons per sq. ft. is allowed. About 1,000 columns are supported on spread footings, the largest of which is 32 by 41 ft. and carries a load of 6,300,000 lb. Tower foundations carry loads from 6,300,000 to 8,900,000 lb. and vary in diameter from 8 ft. 8 in. to 10 ft. 4 in. Minimum pier diameter is 4 ft. and is enlarged only as may be required by limiting stress of 400 lb. per sq. in. in the 1:2:4 concrete (below the spiral) is exceeded. Piers resting on clay have bell bottoms. Foundations cost about $1.25 per cu. ft. and quantities were about 74,000 cu. yd. of concrete and 3,100 tons of reinforcing steel. The paper on foundations outlines the problems and mentions factors such as bearing power, skin friction, subsidence, and cost which entered into their solution.—H. J. GILKEY

Examples of mushroom floors in France. W. CHERRE. *Constructeur de Ciment Armé*, Nov., 1930, No. 134, p. 245-247.—Strasbourg firm "Zublin" has erected numerous mushroom floors in Alsace, including building of the Société Alsacienne de Constructions Méchaniques at Mulhouse. It measures 280 by 82 ft. and comprises 7 mushroom floors. Distance between axes of columns is 23 ft. in longitudinal direction and 26.25 ft. in direction of width. Rectangular slabs measure 23 by 26 ft. and represent considerable spans for reinforced concrete construction, in view of the heavy loading. Loading on ground floor is 1635 lb. per sq. ft. and that of floor above is 410 lb. per sq. ft. Depth of slabs is relatively low, being 20 in. for the ground floor; 10 in. for first floor and 9 in. for floor above. The computations were made in research department of Zublin Co., according to method of Prof. Ritter of the Zurich Polytechnic School. The building was completed in 1926. Another example, built in 1913 by the same company, is warehouse of Société d'expeditions Léon Weiss at Strasbourg. The surface area of the mushroom floors is 86,000

sq. ft.; they are designed to support loads varying from 305 to 450 lb. per sq. ft. The span is 15 ft., the depth of slab varies from 8½ to 9¾ in. Another example is the covering of a reservoir of Dollfus-Mieg Co. at Belfort, built in 1927. The columns are spaced 11.5 ft. in one direction and 11.1 ft. in the other. Depth of slab is about 6 in., designed to withstand a loading of 490 lb. per sq. ft. The author urges further study of this type of construction.—M. A. CORBIN

Continuous slabs reinforced in two directions. W. S. GRAY. *Concrete Constr. Eng.* (England), Dec., 1930, v. 25, No. 12, p. 661-669.—It is often assumed in designing interior panels of uniformly loaded continuous slabs reinforced in two directions that the positive moment is 0.8 times that calculated for a freely-supported panel. There is little justification for this assumption, its derivation being from analogy with beams or slabs reinforced in one direction only. Experience has proved this method is on safe side, but gives no guide to factor of safety. It is well known that method of design of slabs reinforced in two directions by dividing slab into two series of rectangular strips is inaccurate. A more economical design is obtained if reinforcement is designed to resist torsional moments at all points in the slab. Formulae were developed by Dr. H. Marcus which are applicable to mesh-reinforced rectangular slabs with any number of fixed edges and may be adapted to deal with slabs continuous in one or more directions. This method of calculating moments is specified now in German regulations for reinforced concrete. Method of obtaining values of moments is: Slab is assumed divided into two bands of rectangular strips considered as acting independently, and expressions for moments obtained. Assuming equal values of EI for the two strips at right angles by equating values for deflections for each strip values of moments can be obtained for each strip in terms of uniform load and length of two strips. These expressions for moments are multiplied by reduction coefficients if corners are restrained, thus allowing for torsional moments and restrained deflection of strip. Curves are plotted showing relation between values of reduction factors and ratios of sides of slab. Ratios of sides of slab of 0.5 to 2.0 were considered. Curves are plotted for different degrees of restraint. Following cases are treated: slab freely supported, fixed on one side, fixed on two opposite sides, fixed on two adjacent sides, fixed on three sides and fixed on four sides. Method is given of obtaining the moments caused by dead load and live load in continuous slabs extending over several panels. Method used is similar to that of obtaining influence lines for continuous frames with unsymmetrical loading, and considers dead load plus live load in alternate panels. Expressions for moments in interior panels can be obtained using the above curves for interior slabs under loading considered fixed on two sides and simply supported on other two. Values of moments due to live load on alternate panels can be similarly obtained from curves for case of panels simply supported. Combining two sets of values given by two loadings moments due to dead plus live load can be obtained. An example explains method of computation for a six-panel continuous frame. Method has advantage over other methods that it is simple. In other solutions of problem effect of continuity is arbitrarily assumed or treatment is too complicated for ordinary office design. The above paper deals only with point load concentrations. —JOSEPH MARIN

DAMS

Hoover dam. ELWOOD MEAD. *Civil Engineering*, Oct., 1930, V. 1, No. 1, p. 3-8.—The paper outlines the Boulder Canyon Project in its larger and in its preliminary aspects. Nevertheless all aspects of a project that includes a 700 ft. concrete dam, a reservoir eight times larger than the Assuan, machinery to develop a million and a quarter horse power, an All-American Canal to irrigate nearly a million acres with an excavation one fourth that of Panama

Canal all at total cost of $165,000,000, should be of interest to readers of the Journal and it is deemed proper that their attention should be called to this treatment which in itself has little to say about many vital concrete features of the project.—H. J. GILKEY

Hydro power station of Rocky River in Connecticut. *Die Bautechnik* (Germany), Aug. 1, 1930, V. 8, No. 33, p. 510-2.—Hydro power development of Rocky River, built by Connecticut Light and Power Co., consists of a water reservoir covering about 8.49 sq. miles. Capacity of lake is 218.5 millions cu. yd. Earth dam rests on concrete wall founded 5 ft. deep on sound rocks. A canal 3400 ft. long on east side of river leads to intake and power station. A generator, producing 30,000 KW. and 2 centrifugal pumps are installed. Plant was designed for a later addition of a second unit.—A. E. BEITLICH

The "cracked" cantilever in dam design. B. F. JAKOBSEN. *Eng. Con.*, Dec., 1930, p. 439-441.—In a brief historical review of the lack of agreement between calculated and actual deflections of dams, the author discusses overlooked factors which tend to materially increase calculated deflections. Shear deformation, yielding of foundation, effect of swelling of upstream side, and time yield of concrete may all combine to produce deformation ten times greater than that found when only bending, calculated from initial modulus of elasticity, is included. This agrees with present-day theory. References given include L. Jorgensen, Frederik Vogt, W. A. Slater, F. R. McMillan, Aarskog, A. H. Fuller and C. C. More. "Cracked" cantilever and the horizontal "hinge" are treated in detail. Mathematical analyses show that ability of "hinge" to produce more favorable stress distribution is quite negligible. Water entering a crack tends to produce tension at bottom of crack whereas compression might have existed had the crack not been present. Shear is greater due to a crack. Calculations of deflections of dams cannot be made with any degree of precision because the great increase in deformation due to plastic yielding is dependent upon the load history of the dam. Temperature changes and water load on the reservoir may affect the deflection of the dam.—N. H. ROY

Water power stations of river Liri in middle Italy. W. VIESER. *Bautechnik* (Germany), Sept. 12, 1930, V. 8, No. 39, p. 580-3.—Favorable water and geological conditions led to construction of several hydro power stations in course of river Liri between Isola del Liri and S. Giovanni Incarico (Italy). Amount of water is practically constant during almost 6 months. Catching area for upper stations is 586.7 sq. miles, for lower stations 1274 sq. miles. Four stations were built in upper part of river. Structures of power plants in middle and lower part of river Liri include gravity dam in Portefiume which forms a water reservoir of 0.58 sq. miles. Dam is 103.4 ft. high, 262.5 ft. long and has volume of 26,160 cu. yds. Downstream side of dam is protected against force of water by sinking of 27 reinforced concrete caissons, each 36 ft. long and 10.3 ft. wide, in 3 rows. Water canal has cross-section of 35.9 sq. yd. and is capable of delivering 109.9 cu. yd. per sec. Power house is 121 by 33 ft. Hydro power station in Ceprano is fed by two water reservoirs connected by canal. Dam in valley of Liri is of gravity type. 283.8 ft. long, 106.6 ft. high and has 103.4 ft. spillway. Reinforced concrete dam in Rio Cancello is 132.8 ft. long and 72.2 ft. high. It has 3 flood gates, each 6.5 ft. square. Connecting canal is lined with reinforced concrete. Water is delivered to power station through cylindrical reinforced concrete pressure pipe 12.5 ft. diameter. Reinforced concrete power house is 196.8 ft. long. Additional information is furnished of construction of stations. Several small dams, weirs and pressure pipe lines in reinforced concrete were built in Fontana Piri, Fontecupa, Serelle and Liri-Fibreno (Italy).—A. E. BEITLICH

MISCELLANEOUS

Calculation practice for reinforced concrete. "Pratique du Calcul

du Beton Armé." G. MAGNEL. 1930, Van Rysselberghe and Rombaut, Paris (France), 3rd part, 360 pp., 12 tables, Frs. 140. Reviewed in *Bautechnik* (Germany), Sept. 19, 1930, V. 8, No. 41, p. 635.—Main subject of book is theory and calculation of concrete and reinforced concrete arches with 2 and 3 hinges. Investigation is carried out for vertical and horizontal loads.—A. E. BEITLICH

Research advances in civil engineering. ALFRED D. FLINN. *Civil Engineering*, Oct., 1930, V. 1, No. 1, p. 14-16.—Attention is called to the value of research as a means for settling differences of opinion and eliminating waste. The varied agencies engaging in research and a great many specific research problems are mentioned. Among those of greatest probable interest which have been sponsored by the Engineering Foundation in recent years are: research on arch dams and dams in general, concrete arches, earth foundations, earthquake stresses, retaining walls, cements, flow of concrete, testing small models, etc. It is pointed out shat all research should begin in the library.—H. J. GILKEY

Reinforced concrete transverse ties for railroads. ROUDOLF. *Bautechnik* (Germany), Aug. 1, 1930, V. 8, No. 33, p. 507-8.—Cracks occur usually in middle of reinforced concrete ties, made of one single piece, no matter how wide they are. Twin ties show greatest damage. Concrete ties are in general made shorter than wooden ties to decrease their weight. New type of ties, system Roudolf, consists of two parts, 8.8 ft. long, connected by center hinge which prevents cracks. Another advantage is great resistance agzinst horizontal stresses. Railroad ties should consist of not more than two parts.—A. E. BEITLICH

Reinforced concrete design simplified. JAMES R. GRIFFITH. *Concrete*, Dec., 1930, V. 37, No. 6, p. 40-41.—Chart O_1 may be used to design tied columns according to the A. C. I. Code. Chart M_1 may be used in designing spirally reinforced columns according to the Joint Committee or the A. C. I. Code. These two charts complete the series. There are now available design charts for beams, slabs, and columns for 2000, 2500, and 3000 lb. concrete.—N. H. ROY

The proposed underground railway system for City of Rome (Italy) and its reinforced concrete structures. JOSEF STELLINGWERFF. *Osterr. Bauzeitung* (Austria), Nov. 22, 1930, V. 6, No. 47, p. 763.—Extensive underground system is projected and designed for Rome, street traffic of which has increased more than five times during last 10 years. Three main lines are 6.73, 3.31 and 4.91 miles long. For tubes and superstructures reinforced concrete will be used, made of high early strength cement. All structures will be proof against aircraft attacks. (cf. *L'industria Italiana del Cemento* (Italy), May, 1930.)—A. E. BEITLICH

Concrete aqueduct irrigates half million acres. GEORGE P. FRENCH. *Concrete*, Dec., 1930, V. 37, No. 6, p. 39.—A reinforced concrete aqueduct across a dried-up river bottom has been built in the neighborhood of Brooks, Alberta, by the Canadian Pacific Railway. The structure consists of two miles of concrete flume, in basin form, 20 ft. wide by 10 ft. deep, supported by steel and concrete trestles. The maximum height of structure is 50 ft. A large inverted siphon of reinforced concrete carries the water under the railway tracks.—N. H. ROY

Elementary calculation of continuous beams. N. RAUBAL. *Osterr. Bauzeitung* (Austria), Nov. 8, 1930, V. 6, No. 45, p. 735-7.—Formulas are derived and examples calculated which illustrate amount and direction of stresses in different kinds of continuous beams under various loads.—A. E. BEITLICH

Structural design of Chicago's subway. *Eng. News Record*, Dec. 11, 1930, V. 105, No. 24, p. 932-933.—Plans for Chicago's subway have been completed to point of showing how structure is to be built. Essentially, project consists of a four-track subway $3^1/_2$ miles long under State St., extending from Eighteenth St. north across the Chicago River to Chicago Ave. and turning west along Chicago Ave. to Franklin St. In general, the subway consists of a roof and walls of concrete jack arches supported by a structural steel frame and floor of reinforced concrete. The latter is reinforced against an upward hydraulic pressure of 90 lb. per sq. ft. The steel bents are spaced 5½ ft. on centers, and in the station areas they are placed at every third bent or 16½ ft. on centers. The river tunnels 1900 ft. long, consist of one double-track and two single-track tubes. The subaqueous section of the tunnels will be steel tubes sunk in place and lined with reinforced concrete, similar to those in New York under the Harlem River and the LaSalle St. tunnel in Chicago.—D. E. LARSON

ROADS AND PAVEMENTS

New methods of measuring concussions caused by traffic on roads. RISCH. *Betonstrasse* (Germany), Aug., 1930, V. 5, No. 8, p. 194-198.—In general meeting of "Studiengesellshaft fur Automobilstrassenbau" (Institute for the study of the building of automobile streets) the author cited that with increased use of heavy trucks, complaints of damage to buildings and destruction of pipe-lines below the street, claimed to be caused by traffic concussions, have caused desire to measure these concussions to determine truth of these complaints. Instruments used in the past, mostly built according to the principle of the seismometer were not suited. The curves drawn by these instruments were more or less deformed through the vibration of the mass of the instruments. In the "Ambronnschen Beschleunigungs messer" (acceleration meter) is used the Piezoelectricity of closely piled up silicate crystals. The discharge is artificially increased and measured with a galvanometer, actions of which are filmed and the film timed. This instrument is also very easily handled. Several measuring bodies can be placed in different places and directions and the results taken on one film. General principles which have to be taken into consideration in judging results include condition of soil and its ability to conduct concussions.—R. J. DIETRICH

SEWERS

Concrete sewer design adopted as standard after severe test. *Eng. News Record*, Dec. 11, 1930, V. 105, No. 24, p. 937-938.—The city of Richmond, Va., is to construct about 25,000-lin. ft. of large sewer within the next five years. Studies have led to the development of a horseshoe-type reinforced-concrete section equivalent in capacity to circular pipe with diameters from 60 to 144 in. Salient features of the design are: (1) Vertical and horizontal diameters are practically equal to diameter of an equivalent circular section. (2) Invert acts as no part of structural design except where foundations are soft; thus the washing out of invert does not affect stability of arch and walls. (3) Sheathing of trench can be used as the outside form up to springing line. (4) The design is sufficiently flexible to be adaptable to soils of different bearing capacities. As a final check on design, an 8-ft. section equivalent in size to an 84-in. circle was tested by piling pig iron on a platform resting on the upper quadrant of the arch. Hair cracks developed in center of intrados at both ends when load reached 129,419 lb. The ultimate load was about 153,460 lb.—D. E. LARSON

ARCHITECTURAL DESIGN

Reinforced concrete hall designed for clerestory lighting. *Concrete*, Dec., 1930, V. 37, No. 6, p. 34.—Design of Horticultural hall departs somewhat from London building practice in that four tiers of vertical windows rise from the roof arches in step-fashion. Daylight is thus admitted into the large hall by a complete system of clerestory lights. There are several examples of this system on the continent. The building is of reinforced concrete. The foundation rests on clean dry sand.—N. H. ROY

Advantages and versatility of cast stone. WALTER OBST. *Betonwerk* (Germany), Aug. 30, 1930, V. 18, No. 35, p. 487.—Real or imitation, natural or artificial, original or substitute, these are the contrasts encountered in present day enterprises. Many synthetic products today have qualities all their own and do not in the main, imitate the natural product, but in many cases excel. Modern cast stone is such product. It is now used in first-class undertakings, in sculpture and monuments. Cast stone has a structure and texture all its own. It has attained a solid position in the public mind. The outstanding qualities of cast stone are texture, density, color, durability and ease of ornamentation.—H. FRAUENFELDER

Reinforced concrete and aesthetics. E. BLUNCK. *Report of the 33rd main meeting of German Concrete Association*, March 17, 1930, p. 219-35.—Fundamental difference between reinforced concrete and other structural materials is fact that these structures have monolithic character; supporting and supported parts, horizontal and vertical parts are rigidly connected. This construction principle allows greater freedom of shaping. Correct valuation of modern architectural forms is rendered more difficult by traditional opinions, which compare structures with old standards. Uniformity, contrasts, symmetry and rhythm in form and dimensions are most important factors which govern modern aesthetic reinforced concrete structures. Author shows in 15 illustrations selection of recently built structures which are outstanding in their architectural design, and criticises their forms. Such structures are industrial and office buildings, water towers, arch bridges, churches, locks and dwelling houses. Great is number of mistakes and unaesthetic designs, and much fundamental explaining work among contractors has to be done in future.—A. E. BEITLICH

Concrete and reinforced concrete for construction of dwelling houses. MUELLER. *Report of the 33rd main meeting of German Concrete Association*, March 17, 1930, p. 192-219.—Use of concrete for construction of dwelling houses is not very common in Germany on account of its high price and less heat insulating properties than brick walls. Author discusses efforts to overcome these difficulties and describes number of structures recently erected. Concrete with pumice stone as aggregates and air-concrete (large voids) were used for small apartment buildings. More favored is construction with small or larger concrete form bricks, which method eliminates high form costs. Of advantage is shorter time period for drying out of walls. Author describes construction method which uses concrete flat slabs which are especially suited for ceilings forming heat insulating air cells. Reinforced concrete is frequently used for modern apartment house construction allowing greater varieties in architectonical forms. More economic form methods and improved equipment must be developed for further progress.—A. E. BEITLICH

FIELD CONSTRUCTION

BRIDGES

Reinforced concrete bridges of the German railroad concern (Reichsbahn). GEORG MUENZ. *Bautechnik* (Germany), July 11, 1930, V. 8, No. 30,

p. 463-4.—Construction of 3 reinforced concrete arch bridges with spans of 42.6, 49.2 and 82.0 ft. are described and details of their reinforcements are given. Iron portland cement, which gave compressive strength of 7600 lbs. per sq. in. in 28 days, was used in a 1:2½:1½ mix with gravel and broken granite of ¼ to ¾ in. size. Mixing water was kept as low as possible, and concrete was continuously tested during entire construction period.—A. E. BEITLICH

New construction of a viaduct of the Hannover Station in Hamburg (Germany). BLUNCK. *Bautechnik* (Germany), Sept. 5, 26, 1930, V. 8, No. 38-42, p. 563-5, 639-41.—Main railroad line crosses freight tracks in semicircle with radius of 984.3 ft. Old viaduct structure was 393.7 ft. long and rested on masonry piers. Smoke from engines had done considerable damage to railroad superstructure and piers which fact demanded entire reconstruction of viaduct. Solid roadbed is now constructed, on reinforced concrete two-hinged frame construction, made of 1:4 mix. Most all of old piers were provided with reinforced concrete casing. Road slab consists of transverse I-beams, placed 25.5 in. apart, totally encased in reinforced concrete (1:5). Rods (0.7 in. in diam.) are placed above and below beams spaced 12 in. o. c. Protecting layer of 1-in. neat cement and two layers of asphalt and bitumen insulation were placed. Broken rocks of railroad bed are filled on top and between two reinforced concrete embankments which contain pipes for power and telegraph cables. Drafts for engine smoke and drain pipes were built in road slab. Foundations for frame supports were made of 1:6 mix. Conditions were very unfavorable for storage and preparation of structural materials on account of lack of space. Concrete mixer was located on elevated platform. Cement and aggregates were delivered to mixer from small storage bins by elevator. Small traveling cable crane transported concrete from mixer to workmen. Aggregates consisted of 2.5 parts of gravel and 0.5 parts of broken basalt (0.6 to 1 in.). Entire construction program was carried out without interruption of heavy passenger and freight traffic (over 180 trains per day).—A. E. BEITLICH

Southern Pacific Company's Suisun Bay bridge. W. H. KIRKBRIDE. *Proc. Am. Soc. C. E.*, Aug., 1930, V. 56, No. 6, p. 1307-1318.—Double track railroad bridge replaces two train ferrys on main line of Southern Pacific across Carquinez Straits between Benicia and Port Costa. There is possibility that inactive fault plane crosses site and construction was planned to resist earthquake shock by having centers of gravity of piers as low as possible. Specified 28-day concrete strengths were 2500 and 400 lb. per sq. in. in compression and tension respectively. Steel reinforcement at 30 lb. per cu. yd. of concrete was placed to resist accelerations of 5 ft. per sec. which is twice that of the 1906 San Francisco quake and 1½ times the Japanese quake of 1923. Steel superstructures were planned for similar intensities and were notched into heavy monolithic piers. Swift current, deep water and soft mud bay-bottom called for unique foundation methods. From preliminary octagonal pile platform, 81-ft. diameter steel shells were lowered through water into mud from 15 to 30 ft. Shells were constructed of bolted 10 ft. sections and were extended above water level after which they were filled with sand dredged from river. Caissons were then sunk through sand, inside of shell and properly grounded on rock. A 2-yd. floating concrete plant with 104-ft. tower constituted nucleus of concreting equipment.—H. J. GILKEY

BUILDINGS

Farm storage houses—a market for concrete products—ROBERT H. MOORE. *Concrete Products*, Dec., 1930, V. 39, No. 6, p. 14-15.—This article details the construction of two concrete tile farm storage houses, one with cinders and one with Haydite as aggregate. In first one constructed the original

idea was to build it with double walls of 8 by 8 by 16-in. block with a dead air space between. It was found that a wall of this kind would cost approximately 38¢ per sq. ft. The block manufacturer, however, found that a triple wall could be built of 4 by 8 by 16-in. partition tile for about 36¢ per sq. ft. This would not only give better wall with two air spaces instead of one but would also reduce total wall thickness from 18 in. to 16 in. This method of construction was followed on both buildings.—E. S. HANSON

Development of reinforced concrete construction in Finland and introduction of construction control. OTTO WEYERSTALL. *Report 33rd main meeting German Concrete Association*, March 17, 1930, p. 112-37.—One of first great reinforced concrete structures was saw mill in Lauritsala, a two-story building, 185 ft. long and 100 ft. wide with slab floor construction. Span of beams is about 50 ft. Development was very rapidly until 1911, when first construction control was established. Conditions in Finland are very favorable for reinforced concrete construction on account of its great natural sources for sand and clean aggregates. Of disadvantage are climatic conditions, since cold weather periods predominate. Several modern cement plants furnish excellent cements with high early strength properties. Specifications for testing cement and concrete were issued a few years ago and follow in general lines of German specifications. Great number of structures are described and illustrated showing progress made in recent years.—A. E. BEITLICH

Speedy erection of structural frame proves value of inspection. B. L. MEHANDRU. *Concrete*, Jan., 1931, V. 38, No. 1, p. 21-23.—Erection of Indiana hotel structure demonstrated that co-operative effort between architect, owner and contractor benefits all. Commercial inspection engineers were retained. Building is 12 stories high, reinforced concrete structural frame and floors, 83 by 69 ft. in plan, requiring 3,500 cu. yd. of concrete. Concrete mixing plant including ½-yd. mixer was at rear of building. Concrete was mixed in 2-sack batches yielding average of 11.3 cu. yd. Tubular chute distributed concrete direct from light steel frame hoisting tower. Architect's specifications called for 2,000-lb. concrete in floors, 3,000 lbs. for columns in first five stories and 2,500 lbs. above fifth floor. Both floor and column concrete above fifth floor was designed for 28-day strength of 2,500 lb. per sq. in. Proportions selected yielded 5.65 cu. ft. in place, per sack. Water-cement ratio was 7¾ gal. to the sack. Slump averaged 7½ in. Complete field testing equipment, including molds, was used on job. Cylinders intended for 7-day test were cured at job 4 days, then shipped to laboratory and tested. Twenty-eight day cylinders cured same length of time at job, then shipped to laboratory and cured in moist room. Average strengths were 1,790 and 2,785 lbs. respectively. Although concreting operations were started 9 days behind schedule, roof slab was concreted July 18, 9 days ahead of schedule. Concreting work made ordinary progress in first and second floor construction, second floor having been concreted June 5th. Third floor concrete placed on June 11. Thereafter a floor was concreted every Tuesday and Friday, until the twelfth floor was placed July 12. Run from third to twelfth floor equivalent to story every 2½ days. Actual schedule maintained from third to twelfth floors, inclusive, left 1½ days for moving and framing form work. Time required for concreting floor, 8 hours. Period from Monday morning to Tuesday noon was occupied in getting ready for Tuesday concreting (afternoon and evening). Friday concreting preparations were made on Wednesday and Thursday.—C. BACHMANN

How to save in concrete form work. A. B. MACMILLAN. *Concrete*, Dec., 1930, V. 37, No. 6, p. 35-37.—Considering difficulties of establishing necessary bases on which cost of form work may be estimated in advance, best method is to assume certain average conditions and express required form work for given area in terms of material quantities and hours of labor. Reader may then follow similar methods in figuring his form work problems and arrive at es-

timated cost by applying prevailing unit prices. Greatest variation in cost arises from number of times lumber is used. Comparing 2 and 6-story reinforced concrete building of same ground area, in both cases, floor form panels, beam sides and column forms will be made in sufficient quantity to concrete two floors at a time. On 2-story building forms are used once; on 6-story three times. Cost of material for floor form panels, beam sides, and column forms is as much for 2-story as for 6-story building. Experienced designer makes column sizes in 2-story building alike in both stories, reducing reinforcement in upper columns, and dimensions of roof beams and girders (part below the bottom of the slab) same as floor beams and girders, requiring no patching or altering of form work. Similarly in 6-story building, designer will maintain same column sizes from top to bottom and same beam and girder dimensions below bottom of floor and roof slabs. Following points are guide for estimator: (1) For foundation walls and wall footings of building of moderate ground area, provide sufficient form material for entire area. In building having large ground area, where foundation form work can be erected to keep mixing plant going for about three days, put up only part of form work, moving it second day after concreting is completed. (2) For column footings, make up only enough footing forms so that they can be used four or five times. (3) Forms for columns, for sides of beams and girders, for ordinary floor slabs, for flat-slab floor construction, and walls above grade, should be made up in sufficient quantity for two stories if schedule contemplates one story per week. If two stories per week, make up sufficient forms for three stories. (4) Material for posts, jack frames, braces and other material required to support floor form work must be supplied in sufficient quantity for three stories if schedule calls for one story a week; sufficient for four stories if schedule calls for two stories per week. (5) For beam and girder bottoms which are left in place, along with reshoring timbers and braces, much longer than remaining form work. Quantity of material must be sufficient for four stories if schedule calls for one story per week, and for seven stories if schedule calls for two stories per week. (6) Material for rough structural parts of stairways need not be ordered separately, since rough form lumber can be salvaged from material already used.—C. BACHMANN

DAMS

Salt Springs dam. O. W. PETERSON. *Proc. Am. Soc. C. E.*, Aug., 1930, V. 56, No. 6, p. 1319-1334.—Concrete faced, 328 ft. (above foundation) rock fill dam of Pacific Gas and Electric Co., 70 miles east of Sacramento, Cal., is to impound 130,000 acre ft. to develop all economic available power on Mokelumne River. The main concrete feature is flexible reinforced concrete upper facing 1 ft. thick at top and 3 ft. thick at bottom where it joins the concrete cutoff wall. The average slope of upstream face is 1.3 to 1, top width 15 ft., and bottom 900 ft. along axis of stream bed. Crest length is 900 ft. at elevation of 3959. Of 2,900,000 cu. yd. of rock, 220,000 (7.5 per cent) is derrick placed, 15 ft. thick, upon which the concrete facing is laid. The facing is divided into independent 60-ft. squares. Edges rest on concrete supports poured in grooves left in the rock backing. Two layers of 1-in. round bars at 9 in. o. c. each way reinforce lowest layer and this is gradually reduced to one layer of 1-in. sq. bars 1 ft. o. c. at top. Joints between panels consist of soft copper seals embedded in concrete. Concrete 3,000 lb. at 28 days was designed for maximum density with 5 bags cement per cu. yd. Dam is slightly arched in plan to decrease tension in the concrete facing. Cutoff wall is 6 ft. thick and extends from the base of dam 6 to 20 ft. into good rock. Two inch diameter grout holes 50 ft. deep were spaced 6 ft. apart along bottom of cutoff trench where the dam was over 200 ft. high and near sides at lesser height holes were placed 10 ft. apart and reduced to a minimum depth of 25 ft. at top of dam. To provide a more gradual transition from solid rock to loose fill, the bottom 10-ft. of 15-ft. layer of rock backing is laid up in concrete mortar. This should minimize

tendency of slab to crack at junction of facing and cutoff wall. Spillway solid rock excavation was crushed to 2½ in. maximum size and sand is obtained by passing crushed rock through a cone crusher. Each 60-ft. panel is completed in one continuous operation, forms being moved while pouring is in progress. Concrete is spouted into place by chutes from side dumping, 2 cu. yd. cars hauled by gasoline locomotive.—H. J. GILKEY

Foundation treatment of the Rodriguez dam on the Tijuana River, Mexico. CHARLES P. WILLIAMS. *Proc. Am. Soc. C. E.*, Aug., 1930, V. 56, No. 6, p. 1291-1297.—This dam of Ambursen type now under construction may be statistically summarized as follows: Height 187 ft. above stream bed, 240 ft. above lowest foundation bed-rock. Crest length 2,000 ft. Buttress spacing 22 ft. center to center. Buttress thickness from 19 in. to 66 in. Deck thickness 25 in. to 64½ in. Deck slope (lower face) 1:1. Buttress slope (downstream face) 1:5. Spillway capacity 150,000 sec.-ft. Purpose: To irrigate 5,000 acres and supply domestic water for Tijuana (population 10,000). Drainage area 1,670 sq. mi. Reservoir capacity 110,000 acre-ft. Estimated possible flood 150,000 sec.-ft. Foundation exploration: 9 borings and 52 test pits. Selection of Ambursen type was due to variability in bearing power of foundation rock and fact that dam crossed a fault plane. Most unique foundation feature was the construction of arch to transfer part of four of buttress loads to better rock. Arch was flat below and underside rested directly on soft rock in central portion. Construction permits redistribution of stresses as required if softer rock yields. In the fault zone concrete cutoff wall was poured in open cut 26 to 33 ft. below original rock surface (66 to 72 ft. below stream bed). Below these levels excavation was continued where necessary through shafts left in concrete.—H. J. GILKEY

Experiences from a power plant construction in India. CARL THULIN. *Teknisk Tidskrift* (Sweden), Oct.-Nov., 1930, No. 11-12, p. 129-135 and 145-152.—The Swedish contracting company "Vattenbyggnadsbyrån" in connection with Rendel, Palmer and Trittion of London, has just completed power plant construction on Malakka peninsula for The Perak River Hydro-Electric Power Co., London. The power is to be used for the fabricating of tin. The construction was started in Dec. of the year 1926. The plant was located 6 miles from nearest road in a dense jungle, so road was constructed first, involving excavation of 143,000 cu. yd., and building of several bridges. Over 100,000 tons of material, sand and stone not included, was hauled over this road. As much as 3,500 tons of cement could be transported per month. Trucks with solid tires lasted only half as long as trucks with air tires, so during the later part all new trucks were supplied with air tires. Working force consisted of 110 Europeans and as many as 4,500 laborers. The Chinese were used where certain skill was required, as carpenters, lumber men, etc. Tamiles were used for hauling and Bengali for the heavy work. Lascars were used for work high up in the air. An amount of 20,000 English pounds was spent on different methods of fighting malaria, with such a good result, that the number of sick people averaged only 2 per cent, and for long periods as low as .5 per cent. Contrary to customary practice these precautions were not started before start of construction. To connect two sides of river a suspension bridge with free span of 450 ft. was built. It was designed to carry a train weighing 20 tons. The dam was 1,380 ft. long with a maximum height of 143 ft. Total concrete 143,000 cu. yd., steel 5,700 tons, and forms 1,080,000 sq. ft. Pouring of concrete was very difficult on account of the heat, during the middle of the day—about 140° F. The Insley System with two towers 300 ft. high and 1,000 ft. apart was used. The quality of sand and stone varied, and it was necessary to establish a regular concrete laboratory. Best mixture was found to be: cement 460, coarse sand 140, medium sand 240, fine sand 120, big stone 400, and small stone 400. The forms consisted of 2-in. planks and could not be reused many times, because they seemed to wear out very quickly.

Some kind of white ants that ate up all kinds of soft wood did considerable damage to the forms and other construction of wood. The power plant was opened on June 28, 1930.—ODD ALBERT

Effect of shrinkage in large concrete dams. (See PROPERTIES OF CONCRETE.)

MISCELLANEOUS

Concrete benches reduce maintenance costs in greenhouses. *Concrete*, Jan., 1931, V. 38, No. 1, p. 33.—Concrete soil benches vary from 200 to 400 ft. in length. Bench consists of two curbs, each 16 in. high, 2 in. wide at top and 4 in. wide at bottom. Aisle side is vertical and inside battered. Two curbs for entire length of bench are cast in one operation in special steel forms. No joints are provided, cracks are immaterial. Other benches consist of precast 1-in. reinforced slabs, 12 in. high. Eight ft. lengths are used for sides and 2 ft. 6 in. lengths for ends. Cast iron angles driven into ground at corners hold slabs in position. Slabs made in steel forms using 1:2 mortar, reinforced with ½-in. galvanized mesh. Sixty slabs are required for each bench.—C. BACHMANN

Application of construction methods with sliding forms. KNEES. *Report 33rd. main meeting German Concrete Association*, March 17, 1930, p. 388-409.—After giving a short historical review of development of concrete and reinforced concrete construction methods, author describes various methods for concreting with movable and sliding forms which are in use in several European countries. System Koethenbuerger is based on working in two zones with alternating use of 2 like form sections. It is especially suitable for bin construction. Similar, but superior in many aspects is system Heine, which was developed in United States. Sliding forms became more in favor in Europe during recent years. Greater uniformity of concrete, better construction control, no danger of separation of aggregates or formation of voids, monolithic structure, savings in form work and easy operation are few of advantages mentioned. System MacDonald, also originated in the United States, is described and a number of silo structures and chimneys show its great field of application. In Holland and Hungary, Spindle method enjoys great appreciation, where storage bins of enormous dimensions were built. High early strength cement was used in most every case. Detailed description is given of construction of a cooling tower, elevator tower in Rotterdam (Holland), clinker silos in Neuwied (Germany), oil seed silo in Emmerich, chimney in Kiel and several cement storage silos showing construction progress with above discussed methods. (cf. Knees, *Zement* (Germany), June, 1930, V. 19, No. 24, p. 561-5; Langenbeck, *Ibid*, Feb., 1930, V. 19, No. 6, p. 127-9; Hoffmann, *Ibid*, Aug. 28, 1930, V. 19, No. 35, p. 828-34; *J. Am. Concr. Inst.*, Sept., 1930, V. 2, No. 1, Abstr. Sect., p. 24: *Ibid*, May, 1930, V. 1, No. 7, Abstr. Sect., p. 102.)—A. E. BEITLICH

Reconstruction of quay system in harbor of Memel. NIEBUHR. *Bautechnik* (Germany), Aug. 15, 1930, V. 8, No. 35, p. 527-8.—Increasing traffic during last years in harbor of Memel demanded extension of quay to 1115 ft. and excavation of harbor to a depth of 26.3 ft. and later to 32.8 ft. Underground consists of large granite rocks embedded in very fine gray marl. Steel sheet piling of Larssen type, 51 ft. long, were driven and rigidly connected with existing quay wall. Concrete filling between both was supported by 30 ft. wooden piles. Storage house of reinforced concrete skeleton construction, 262.5 ft. long and foundations for tracks of unloading traveling crane were carried out in record time.—A. E. BEITLICH

Subway station Frankfurter Allee in Berlin (Germany). *Bautechnik* (Germany), Aug. 1, 1930, V. 8, No. 33, p. 508-10—In construction of about 1.1 mile extension of subway railroad system of Berlin, excavating and scaffolding

were carried out under great difficulties to avoid interference with street traffic. Only 33 ft. width of construction site was available for transportation and mixing of raw materials and distribution of concrete. Two mixers with one yd. capacity each handled total of 78,500 cu. yd. of concrete. Floor of tunnel consists of several layers of reinforced concrete (1:5) with a number of protective coatings of asphalt and bitumen. Gravel was especially prepared and did not contain more than 40 per cent passing 4 mesh sieve and left a residue of at least 30 per cent on 8 mesh sieve. Clay content was below 1.5 per cent.—A. E. BEITLICH

Extension of a quay in the harbor of Reiherwerder (Germany). SCHULZE. *Bautechnik* (Germany), July 25, 1930, V. 8, No. 32, p. 487-91.—Quay is 502 ft. long and supported by one row of flat and three rows of round reinforced concrete piles. Total of 989 piles were driven. Mixture of 1100 lb. blast furnace slag cement and 38.84 cu. ft. gravel with a w/c-ratio of 0.50 was used for manufacture of these 50-ft. piles. Pile heads and ends of reinforcements penetrate about 16 in. into main concrete flat slab of quay. Its bottom layer is made of mixture of 666 lb. iron portland cement with 38.84 cu. ft. gravel, while 550 lb. iron portland cement with 42.37 cu. ft. gravel was used for top layer. Joints between flat concrete piling were filled with fabric hose into which a 1:3 cement mortar was poured. Two coatings with Inertol protect concrete against aggressive waters. Foundations of tracks for traveling ore crane rest on vertical reinforced concrete piles placed in distances of 7.2 ft. Iron portland cement was used exclusively for their construction. During entire construction period all structural materials were carefully examined, also compressive strength of concrete and control of mixing operation and distribution. Moisture of aggregates was determined daily to hold w/c-ratio of 0.50 constant. Concrete was mixed in 17 cu. ft. mixer and distributed from movable platform through wooden chutes.—A. E. BEILTICH

Wolfe's Cove Terminals Construction. C. A. LEIGHTON. *Canadian Engineer*, V. 59, No. 21, Nov. 18, 1930, p. 637.—Wolfe's Cove Terminal is being built by Port of Quebec authorities, City of Quebec, in connection with harbor development in the St. Lawrence River. Project covers river frontage of city proper for a distance of two miles, giving total wharfage of 26,000 ft. Construction consists of quay wall for entire length with piers at intervals, which are 2300 ft. long and 435 ft. wide, extending out into river. Wolfe's Cove Terminal consists of 4300 ft. quay wall made up of timber crib to low water supporting superimposed concrete retaining wall. Cribs are 42 ft. high, 62½ ft. wide at bottom and 30 ft. at top. Concrete wall is 26 ft. high, 19 ft. wide at base and 6 ft. wide at top. Concrete was poured one year after building cribs, to allow time for settlement. Concrete face in contact with water up to mean high tide is faced with pre-cast concrete. Pre-cast units are molded in steel forms and cured with live steam. They are placed in the work after two weeks storage. Concrete mix is 1:1½:3. Concrete backing consists of 1:3:5 concrete of which about 50 per cent is placed through tremie. Top of wall above mean high water consists of 1:2:4 concrete. Quantities required are 6500 cu. yd. pre-cast and 50,000 cu. yd. mass. concrete.—G. M. WILLIAMS

Concrete linings for canals—experiences and improvements. S. KURZMANN. *Bautechnik* (Germany), Sept. 5, 1930, V. 8, No. 38, p. 568-71.—Concrete lining has in general 3 conditions to fulfill. It must (1) protect dams and bottom of canals against attacks of water and provide canal with permanent cross-section, (2) decrease roughness of walls and prevent losses of water and (3) protect adjacent land against excessive water infiltration. Observations were made of behavior of canals of several great Bavarian hydro power stations and experiences and improvements are reported. It was found that expansion joints were widened causing some water loss. No great destruction by frost could be found when concrete was properly placed in not too

lean a mixture. Concrete of a 1:0.7:16 cement-trass mixture had partly failed and was replaced. When water level in canal changes frequently, it is possible that great voids are formed underneath lining by washing away of fine material through joints or small leaks. Collapse of concrete walls results. This can be prevented by placing layer of gravel underneath concrete. Special consideration was given to careful treatment of finished concrete. Sand (0 to 0.2 in.) and gravel (0.2 to 1.2 in.)were used in proportion of 2:3 with 300 lbs. cement for 35.3 cu. ft. concrete. Very uniform and dense concrete was obtained. Surface was kept wet for 5 days and covered with ½ in. boards for 2 days. Then a thin suspension of 1 part of cement and 2 parts of very fine sand in much water was poured over concrete to fill fine haircracks and smoothen surface. Additional information is furnished concerning concreting methods, equipment for such jobs, their capacities, power consumption and economy.—A. E. BEITLICH.

ROADS AND PAVEMENTS

Concrete road construction in Argentine. C. E. REYNOLDS. *Concrete Constr. Eng.* (England), Dec., 1930, V. 25, No. 12, p. 691-697.—In describing building of concrete roads in Argentine following topics are treated: The design of roads including the shape of sections used and methods of construction; specifications for materials, including size of ingredients, proportions used, and strength of material; mixing and placing of concrete; method of control of water content by "slump tests"; description of the types of expansion joints used; details of method used in reinforcing concrete.—JOSEPH MARIN

Settled concrete road slabs lifted and leveled by force pump. *Concrete*, Jan., 1931, V. 38, No. 1, p. 27.—Settled concrete slabs are being raised and leveled by pumping mixture of loam, cement and water under them. Pump has chamber in which materials are mixed. Mixture is forced through 2-in. hose. Holes slightly smaller than outside diameter of the hose are bored in the settled pavement on about 6-ft. centers. Hose is forced into one of holes and pump started. Swelling of hose by pressure of mixture holds end fast in hole. Pressure is 5 lbs. or less per sq. in. Mixture hardens in 10 to 12 hours.—C. BACHMANN

SHOP MANUFACTURE

Piece-work vs. day labor. *Concrete Products*, Dec., 1930, V. 30, No. 6, p. 34-35.—This symposium was participated in by 62 readers of whom 14 or 22½ per cent are using the piece-work system in some form or other. Its customary application is only to the men who are operating machines producing block, roofing tile, etc., but occasionally it is applied to other men throughout the plant.—E. S. HANSON

An excellent vibrating apparatus, the table vibrant. JACQUES DUPY. *Rev. materiaux construction trav. publics* (France), Nov., 1930, No. 254, p. 458-9.—An apparatus for mechanical vibration of concrete products requiring small power. The amplitude of vibration can be regulated. Machines of 880 and 4,400 lb. capacity are described.—F. O. ANDEREGG

Gelatine moulds. *Conc. Building Conc. Products* (England), Dec., 1930, V. 5, No. 12, p. 165.—In preparation of the gelatine for casting concrete figures, gelatine sheets should, if new, first be placed for 15 minutes between wet sacking to make them pliable. They should then be heated in a steam jacketted pan until the gelatine is just hot enough to pour, usually after about six hours, gentle heating. All models should first be varnished with shellac varnish and oiled with colza oil. The moulding may be used for about ten times. For reuse it should be washed and cut into 1-in. cubes and melted, without the addition of water, in the pan. New gelatine may be added to freshen up the old.—JOHN E. ADAMS

Operator utilizes excess pea gravel in manufacture of concrete blocks. *Pit and Quarry*, Dec. 3, 1930, V. 21, No. 5, p. 46-48.—Simple outdoor equipment has been installed by Shera Sand & Gravel Co., at its 300-ton plant near Richmond, Ind. A survey indicated possibility of selling locally as many concrete blocks as usual plant crew could produce during the usual idle periods. This has proved correct and program will be continued as it assures year-round employment. Operations include excavation, screening and washing of sand and gravel. Block machine produces 35 blocks per hr., each 8 in. by 8 in. Concrete is prepared in a worm-type continuous mixer. During past summer stock of pea gravel was maintained at minimum by this practice.—A. J. HOSKIN

Material producers making pipe. DUDLEY W. MOORE. *Rock Products*, Dec. 6, 1930, V. 33, No. 25, p. 89.—New plant of the Oklahoma Cement Pipe Co., Tulsa, Okla., is owned by Price Sand Co. and Hughes Stone Co., is also connected with Builders Concrete Co., which sells ready mixed concrete. The new pipe plant acts as an auxilliary plant for ready mixed concrete when this is desirable. Pipe plant produces 60 to 70 tons per day, largely storm sewer pipe handled by 2½-ton electric crane. An 800-ton octagon steel storage tank holds pipe materials. It has five compartments for stone and sand and cement compartment of underslung type, all fed by single elevator supplied from track hopper by an apron feeder. Bins are discharged by gravity to a 7-yd. weighing batcher. Batched material goes by conveyor system to a 21-cu. ft. mixer and heavy duty pipe making machine. Water is added from a measuring tank. Pipe is molded on a revolving table and then core is pulled by small electric hoist and molded pipe is sent to the curing room. Sizes are from 12 in. to 60 in. dia., sizes above 18 in. being reinforced. Curing is by placing pipe under water sprays for five days after which it is stored in the yard. All machines have individual motors with inclosed gear reducing units.—EDMUND SHAW

New concrete brick factory in Surrey. *Conc. Building Conc. Products* (England), Nov., 1930, V. 5, No. 11, p. 144.—An up-to-date factory for the manufacture of concrete bricks, is described, where bricks of all standard shades are made. Only mineral oxides are used as coloring agents, so there is no risk of fading. For partition walls, bricks are mixed 10:1 while 8:1 mix is used for external wall bricks. For greater strength 4:1 or 3:1 bricks are made. These bricks can be easily cut in any plane, and although they prevent penetration of moisture, they do not cause condensation. The materials are raised by a pump through a pipe line to three large settling bins beside the supplying lake, and to assist the raising, charges are exploded below water twice weekly. From the bins the sand is taken by jubilee skips to factory. Sand is cream colored, while cement and coloring material are ground together in ball mills specially designed. The proportions for the various colors are as follows: Red, 8 per cent of brick red oxide of iron by weight of cement; dark red, 8 per cent crimson red oxide; dark brown, 4 per cent crimson oxide plus 2 per cent mineral black; light brown, 4 per cent brick red plus 1 per cent mineral black; yellow, 5 per cent yellow ochre; tan or biscuit, 1¼ per cent brick red oxide; light stone, 2 per cent yellow ochre. The mixers used are of the pan type with two rollers which just clear the bottom of the pan, of 4 or 5 ft. capacity and turn out each about 1200 bricks per hour. The mixture is only sufficiently wet to bind if squeezed in the hands. From the mixer the materials are elevated to hopper over a revolving cylinder carrying three brick molds into each of which correct amount of concrete is rammed. Pallets are passed from discharge end of machine to steel racks and thence to curing shed. Bricks are kept moist for 48 hrs. At first they are sprayed with moisture to avoid pitting, then racks are covered and well saturated for rest of the 48 hours. For transporting to covered storage, portable roller conveyors with adjustable stands are used.—JOHN E. ADAMS